ACI News Letter

Vol. 23, No. 8 JOURNAL of the AMERICAN CONCRETE INSTITUTE April 1952

PREPAKT CONCRETE

INTRUSION GROUT

FOR

CONCRETE AND MASONRY REPAIRS • NEW CONCRETE CONSTRUCTION • BRIDGE PIERS • COFFERDAM SEALS • CAST-IN-PLACE PILES • CASING AND PROTECTION OF WOOD PILES • GROUTING • FOUNDATION STABILIZATION • UNDERPINNING • TUNNEL MAINTENANCE • DAMS • RETAINING WALLS • GROUTED CORE WALLS • EXPLORATION

Years of PREPAKT experience and careful job control assure you of the following qualities in PREPAKT CONCRETE:

- Any desired ultimate strength—1500 to 7000 p. s. i.
- No setting shrinkage and 50% less drying shrinkage.
- High permanent bond to existing concrete.
- Superior weather resistance, especially to cyclic freezing and thawing.
- Superior resistance to action of salt water.
- High impermeability to moisture.

PREPAKT, with all these favorable properties, requires 30% to 60% less portland cement than ordinary concrete of equal strength. PREPAKT may be made under water as well as in the dry, with equal facility and without increasing the cement content.

CONTRACTORS ENGINEERS

PREPAKT

SPECIAL SERVICES

INTRUSION-PREPAKT, INC. THE PREPAKT CONCRETE CO.

CHICAGO • TORONTO • SEATTLE **CLEVELAND 14, OHIO** SAN FRANCISCO • PHILADELPHIA

ZURICH • PARIS • MADRID • STOCKHOLM • HELSINKI • BERLIN

ACI
News Letter Contents

April 1952

On the Cover—A. T. Goldbeck, President of the Institute for 1952. Mr. Goldbeck, Engineering Director, National Crushed Stone Assn., has been active on Institute administrative and technical committees since 1922. He is also the author of a number of technical papers in the ACI Journal.

1952 ACI Convention Story 3

Who's Who

Harry F. Thomson.....29
Johannes Andersen and Poul Nerenst.....29
T. V. D. Woodford.....29
I. E. Morris.....29
J. D. Lindsay and H. W. Russell.....32
Stanton Walker, D. L. Bloem and W. G. Mullen.....32
Arthur R. Anderson.....33

Positions and Projects—ACI Members.....35

New Members.....41

Honor Roll.....42

Tools, Materials, Services..42

List of Advertisers.....46

Cincinnati meeting has wide appeal

Cincinnati was a congenial host to 654 members and visitors at the 48th annual convention of the American Concrete Institute held in the Netherland Plaza Hotel beginning the 26th, and ending the 28th of February.

The convention was one continual round of sessions, committee meetings and luncheons from early morning till late evening. The technical program, one of the broadest ever attempted, was accomplished by holding concurrent sessions Wednesday morning and afternoon, making it possible to present a good variety of subjects in the limited time available. Papers which have not already appeared in the ACI JOURNAL will be published in later issues.

At a luncheon Wednesday noon the Institute, through President Thomson, presented its yearly and special awards. Raymond E. Davis was honored by the Henry C. Turner Medal "in recognition of numerous outstanding and greatly diversified contributions to the technology of concrete as a material." This was the first time since 1947 that this award had been presented. "For his many years of quiet perseverance in directing and co-ordinating development of standards of practice for reinforced concrete design," Douglas E. Parsons received the Alfred E. Lindau Award. The Leonard C. Wason Medal for the most meritorious paper was presented to Walter H. Price for his paper, "Factors Influencing Concrete Strength." W. J. McCoy and A. G. Caldwell received duplicate Wason Research Medals for their paper entitled "New Approach to Inhibiting Alkali-Aggregate Expansion." Otto Safir who received the Construction Practice Award for his paper "Precast Concrete Construction in Canada" was unable to attend the convention; his award was mailed to him.

Continued on p. 4

Continued from p. 3

An innovation which also took place at the Wednesday luncheon was the presentation of certificates to the retiring members of the Board. Recipients were Harmer E. Davis, Harry Delzell, N. M. Newmark, R. W. Spencer, Past President Robert F. Blanks, and Harvey Whipple for his 32 consecutive years of service.

New officers announced after the report of the tellers are A. T. Goldbeck, president; C. H. Scholer, vice-president; and A. Allan Bates, J. W. Kelley, F. N. Menefee, and Bailey Tremper, directors.

Subjects of the technical sessions were Prestressed Concrete, Durability, Curing, Ultimate Load Design, Materials and Properties of Concrete, Design and Construction. Thursday morning was devoted to the annual open session of Committee 115, Research, with Stephen J. Chamberlin presiding. This yearly session gives ACI members an opportunity to preview some of the future expectations for concrete. Thursday afternoon Professor Gilkey held the reins of the Panel Session and directed the efforts of his, as he put it, "so-called experts."

Thursday's highlight, however, was the Harvey Whipple Luncheon. About 200 friends participated in honoring a man whom they had come to know and respect in the 32 years he served the Institute. Stanton Walker was chairman for the luncheon and Professor Gilkey gave the address thanking Harvey Whipple on behalf of ACI members and friends. Mr. Whipple was presented with a radio-phonograph combination purchased by donations from his many friends. Mr. Whipple replied briefly stressing the human idea of the Institute, "the Institute is after all—folks," and he pointed to the future we must look forward to in view of the steadily increasing membership. He closed his talk with sincere thanks to the many with whom he has worked during his long and satisfying career as Secretary-Treasurer of ACI.

The Institute will continue to make use of Mr. Whipple's journalistic experience as editorial consultant for the Journal. His former task as editor will be taken over by Fred F. Van Atta who is also acting secretary-treasurer of the Institute. William A. Maples, former associate editor, becomes managing editor.

FIRST GENERAL SESSION

President Harry F. Thomson called the Convention to order promptly at 10:30 a.m. Tuesday.

The first session included approval of Bylaws changes, appointment of tellers and two papers on prestressed concrete design.

Bylaws changes

The proposed changes result in clarification of the definitions of membership grades and details the method for electing Honorary Members. They also remove the Secretary-Treasurer from the Board of Direction and eliminate the necessity of reappointing him annually. The remainder of the changes are editorial. The revisions were approved unanimously subject to ratification by letter ballot. The committee for Bylaws changes was composed of Professor Gilkey, who presented the motion for their adoption, Charles E. Wuerpel and A. T. Goldbeck.

Hunter W. Hanly and Julian B. Carson were appointed tellers to canvass the ballots for the election of officers.

Prestressed Concrete

Prestressed and conventional reinforced concrete design compared

Professor G. C. Ernst in conjunction with C. O. Brunken and A. R. Riveland, all from the University of Nebraska, Lincoln, reported an exhaustive study of eight different stress combinations for conventional and prestressed concrete reservoirs in an effort to discover the most economical combinations. The studies included 180 complete designs and 80 additional wall designs. They found that prestressed tanks have a lower first cost than conventional designs if the unit cost of the prestressed wall in place does not exceed approximately twice that for a

conventional wall. They further determined that with capacities less than 2.5 million gallons, above-ground reservoirs are cheaper if the controlling water elevations are the same and if there is an unrestricted site location.

Substantial savings in material, up to 61 percent in steel and 35 percent in concrete, were possible with the prestressed above-ground tanks.

Parallel beam construction—Michigan's first prestressed concrete bridge

Clair L. Johnson of Johnson and Anderson, Pontiac, Mich., discussed the use of parallel prestressed beam construction using I-shaped blocks. In effect, it made the bridge a hollow-cored slab with longitudinal and transverse prestressing. Tensioning of the bridge was done in three stages. As the girders were assembled, slight prestress was applied to insure a snug fit. Second stage tensioning followed immediately after the grouting. Approximately 5000 lb per girder was applied, which fell off almost immediately. Final tensioning by means of hydraulic rams was done after the grout had attained final set.

Mr. Johnson explained that this bridge was in the nature of an experiment and that there will be many improvements in future designs. For example, future bridges will have the top flange of the beams narrower than the end blocks and bend-up blocks. This will leave a longitudinal opening about 2 in. wide and this will be filled with concrete to the bottom of the top flange, when the slab is placed. Transverse prestressing strands will run through notches in the blocks. This will simplify transverse prestressing and will aid in producing a composite effect.

In the discussion which followed the question of block quality and the effect of bad blocks was brought up. Mr. Johnson replied that an extremely close inspection of the blocks was made as they were produced. This, coupled with testing some of each group of blocks that came to the job, aided in establishing a uniform block quality. Johnson replied also that using their system meant that the destruction of one block did not necessarily mean the destruction of the whole beam as one block could be replaced easily. Another question was how transverse distribution of the load was attained. Mr. Johnson said that this was provided by transverse prestressing which aided in distributing the load from one girder to another. Also, the reinforced wearing surface helped to some extent in the lateral distribution of the load.

DURABILITY AND CURING

Survey of railroad facilities indicates maintenance costs for concrete structures high. Research made into causes of concrete deterioration. Pavements and hydraulic structures can be durable.

Committee 612 presents round table on curing concrete. Curing methods survey reported. Pigmented membrane curing of canal linings.

Maintenance costs average $964,000

A survey covering a five-year period, from 1942-1946 inclusive, of 17 railroads was summarized by C. B. Porter and reported at the Wednesday morning durability session by Walter Schakel, engineer of structures, New York Central Railroad. It showed that the average maintenance cost for reinforced concrete structures was $964,000 for the five-year period. Many of the present structures requiring costly maintenance were built around 1905, "the wet mix era." With the innovation of the water-cement ratio a period of education began and eventually this new concept of concrete design came into its own. However, it was accepted as a cure-all and not until a real research program was adopted did scientific concrete placement and design gather momentum. This last is illustrated by the survey dealing with some of the later structures which show signs of being a good deal more durable than their rule-of-thumb predecessors.

Alkali-aggregate expansion

An investigation into the specific causes of concrete deterioration was reported by R. W. Gilmore, Baltimore and Ohio Railroad, Cincinnati, Ohio, in the nature of a review of the fourth progress report made by Committees 8 and 29 of the American Railway Engineering Association. The first progress report stated that the cause of deterioration was use of too much mixing water, the second, use of deleterious aggregates which caused pop-outs, the third, excessive seepage of water from embankments, and the fourth cause investigated was volume changes due to shrinkage, unequal expansion of concrete due either to volume changes of the aggregate because of temperature change, or to alkali reactions.

Investigation of several reinforced concrete structures proved conclusively that alkali-aggregate reaction had taken place and caused deterioration and loss of strength. The report discussed the structures in a specific manner and made several recommendations for avoiding this type of deterioration. Thorough analysis of all sand and fine aggregates to be used in a mix, use of low-alkali cement and finally substitution of about 20 percent pozzolan were major remedial measures advised.

Air entrainment prime factor in concrete durability

Freezing and thawing tests of concretes with varying water-cement ratios show that the sonic modulus in air-entraining concrete is only reduced by a maximum of five percent in 384 freezing and thawing cycles compared with 60 percent maximum for concrete without air-entraining agents. That is, the worst water-cement ratio specimen of the air-entraining group loses only five percent sonic *E* whereas the best of the non-air-entraining group loses 40 percent. Therefore, said Frank Jackson, Principal Engineer of Tests, U. S. Bureau of Public Roads, "My laboratory and field experience with air-entraining concrete have convinced me that we may ignore water-cement ratio (at least within the usual working range) insofar as its effect on durability is concerned, in most types of structural concrete—that is, concrete which must be designed for strength."

Durability calls for sound construction practice

Lewis H. Tuthill, U. S. Bureau of Reclamation, Denver, discussed methods other than the use of air entrainment, pozzolans and Type V cement, which will add to the durability of hydraulic structures. He listed good drainage, ample coverage, well-made construction and contraction joints, reduction of leaching, good alignment of surfaces, factors other than Type V cement for sulfate resistance, control of temperature differentials in mass concrete, utilization of vibration to place better concrete and just plain good construction practice.

Permeable concrete in hydraulic structures

Byram W. Steele, consulting engineer, Miami, Fla., struck at the deterioration of hydraulic structures from the standpoint of permeability. He pointed out that the vicious circle of infiltration and consequent loss of bond is the direct result of permeable concrete. He detailed some ways in which the condition might be relieved.

Rostrum is neglected momentarily for the camera during one of the technical sessions

Curing—A current problem and what to do about it

Introducing the curing session, A. G. Timms, Bureau of Public Roads, compared early with present-day curing methods and described two general methods of curing: (1) those that supply water throughout the hydration process and tend to maintain a uniform temperature (ponding, sprinkling, application of wet burlap or cotton mats, wet earth, sawdust, wet hay or wet straw) and (2) those designed to prevent loss of water but of little influence in maintaining the uniform temperatures (waterproof paper or impermeable membrane). A third possible method is a combination of the first two.

Survey of curing methods

A limited survey of curing methods used by state highway departments was summarized by D. L. Robinson, senior engineer, Bureau of Materials, Missouri State Highway Dept. The most widely specified method of initial or preliminary curing is with saturated burlap. For final curing, covering with saturated cotton, felt or jute mats is the method most generally permitted. Covering with saturated hay or straw is the least permitted final curing method.

Mr. Robinson reported fairly good agreement between the states as regards initial and final curing methods. However, wide discrepancies were found in the curing time required. He recommended that a more accurate and enlightening report on current practice be undertaken by securing more detailed information from the various highway departments.

Internal temperature presents problem in curing mass concrete

According to Roy W. Carlson, consulting engineer, Berkeley, Calif., and W. R. Johnson, senior materials engineer, Corps of Engineers, Portland District, curing mass concrete involves about the same fundamental principles as curing other type of concrete. However, the effects of internal concrete temperature is an additional factor to be considered. This may still be rising at the center, while near the face the temperature is falling. If the differential in temperature between the inside and outside increases too rapidly, and the concrete has not gained sufficient strength to resist this change, cracking may occur. The use of steel forms, limiting the height of lift to a maximum of 2½ ft and cooling of concrete before placing, or use of embedded cooling systems, help to overcome this difficulty.

The authors summarized recommended curing practices for mass concrete based on the most recent report of Committee 612.

Canal linings cured with liquid membrane

G. E. Burnett, materials engineer, Bureau of Reclamation, Denver, Colo., discussed the use of a liquid membrane sealing compound for curing concrete in canal linings. This method is particularly useful where a shortage of water might exist, such as an irrigation canal in the desert.

In the past years, many curing methods have been examined and tested and the use of a liquid membrane curing compound has been adopted. The materials themselves

WANTED

ASSISTANT EDITOR

JOURNAL OF THE AMERICAN CONCRETE INSTITUTE

Recent Civil Engineering Graduate preferred with experience on staff of college magazine or newspaper.

State salary expected and send appropriate personal, educational and experience data to

Acting Secretary-Treasurer

American Concrete Institute 18263 W. McNichols Rd. Detroit 19, Michigan

have evolved from the original black bituminous substances to clear liquids and finally to white or light gray pigmented compounds. Mr. Burnett reported definite advantages in using pigmented compounds. They facilitate inspection for proper coverage, maintain lower concrete temperatures which aid in reducing expansion and contraction stresses, and lower the water vapor pressure within the concrete which decreases evaporation.

Generally, the sealing compound is sprayed onto the concrete promptly after finishing but not until free moisture has disappeared from the surface. Laboratory tests indicate that the optimum time for application is just after the concrete attains its initial set.

Economy and practicality goal in curing specifications

H. C. Vollmer, formerly research engineer, National Bureau of Standards, discussed various curing methods and their advantages, disadvantages and application. Economic considerations rarely permit ideal curing methods to be used in the field. Current specifications are generally a compromise between the ideal and the economical, that is, methods which enforce minimum standards for performance and safety. He concluded by saying that "All curing methods should be applied immediately after casting or placing portland cement concrete and should continue without interruption during the early critical periods of optimum value in strength, durability and economy are to be realized."

He also emphasized the need to bring laboratory knowledge to the men in the field. More widespread information on practical methods is necessary if we are to take advantage of what we have already learned.

Curing problems in building construction

Prof. H. J. Gilkey, Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames, summarized the factors which relate especially to the curing of concrete in building construction. He pointed out where and how these differ from some of the problems encountered in other uses of concrete and summarized current practices, especially with regard to form removal and to temperature and moisture control under cold weather conditions.

ULTIMATE LOAD DESIGN

Basic concepts and philosophy of ultimate load. A discussion of load factors and design.

Ultimate load theory offers more realistic concept of design

The symposium on ultimate load design opened with an introduction by Leo H. Corning, manager, Structural and Railways Bureau, Portland Cement Assn., Chicago, Ill., who said that had this session been held 50 years ago a much larger percentage of the audience would have beerr familiar with the subject to be presented.

Around 1900, men such as Ritter, Talbot, Withey and Mensch based their design theories on the actual behavior of the materials with which they were working. In this manner they maintained a good correlation between test results and calculated values. Since then much design has been based on arbitrary rules virtually unrelated to actuality.

Corning said that ultimate load design is based on equating the strength of a section to the stresses produced by the ultimate load which is some multiple of the maximum load to which the structure will be subjected. It will be considerably in excess of service loads and will exceed with an adequate safety factor the maximum load which might be expected at rare intervals or unexpectedly. That is, plastic action at the ultimate load will be recognized but at no time will the structure be designed for working loads outside the elastic limit.

In recent years several countries have permitted the use of ultimate load design and Russia has made it mandatory. In some cases it is permitted as an alternate method as in Brazil and Czechoslovakia, and several large structures there have been designed by this method. Savings up to 20 to 30 percent in steel have resulted without increasing allowable working stresses. Therefore Mr. Corning claimed there is ample precedent for a method which promises a more real concept of design.

Fundamental concepts of ultimate load design

Eivind Hognestad, research assistant professor of theoretical and applied mechanics, University of Illinois, Urbana, dealt with the historical background and basic philosophies of ultimate load design, and discussed the properties of the materials and applicable design equations.

Professor Hognestad concluded that the application of the ultimate load theory provides simplification of design equations as compared to the straight line theory, and a design based on facts which have become available through test and research. He stressed the need for a compilation of current thought to produce a generally accepted procedure for design.

Why design by ultimate load theories

Boyd G. Anderson, associate partner, Ammann and Whitney, New York, N. Y., listed several reasons why structures should be designed by ultimate load theories: (1) The design of all concrete members, subject to any type bending, with or without compression steel would be combined in one common rational procedure, (2) a uniform factor of safety may be provided throughout the structure, (3) ultimate strength formulas anticipate compression reinforcement acting at yield point stress and (4) a simplified approach to the design of all members.

Research on ultimate strength of members

C. P. Siess, research associate professor of civil engineering, University of Illinois, Urbana, described tests to determine the manner of failure and ultimate strength of reinforced concrete structural members. Emphasis was placed on tests in the United States, but foreign tests were also mentioned. Available test data were reviewed and evaluated and the additional data needed was considered.

Practical application of ultimate load theories

Practical application of ultimate load theories to the design of sections was demonstrated by Raymond C. Reese, consulting engineer, Toledo, Ohio. Designs were developed for slabs and beams and for columns, with and without bending, by the conventional and ultimate load design methods. The interest aroused could not be satisfied in the limited discussion period and many assembled at a special evening conference on ultimate load design.

Load factors

T. Y. Lin, University of California, Berkeley, compared load factors in ultimate design and allowable stresses in conventional design. The basis for the determination of load factors was considered and tentative recommendations for their selection were made.

MATERIALS AND PROPERTIES OF CONCRETE

Thermal coefficient of expansion related to durability. Effects of calcium chloride and Chicago fly ash discussed. Bond in welded wire fabric a form of anchorage.

Thermal coefficients of expansion and durability

The effect of thermal coefficients of expansion of aggregates on durability of concrete was reported by Edwin J. Callan, chief, Thermal Research and Field Durability Section, Concrete Research Division, Waterways Experiment Station, Corps of Engineers, Dept. of the Army. His investigation showed a reduction in durability when the differential expansion of mortar and coarse aggregates increased. A difference between the thermal coefficient of cement and aggregates greater than 3.0 x 10^{-6} per degree F suggested caution in choice of the aggregate combination; further testing by freezing and thawing prior to acceptance was recommended.

In the discussion that followed, Stanton Walker pointed out that thermal shock occurs in the cooling cycle and not in the warming cycle in freezing and thawing tests. This introduces tensile stresses which are not likely to occur in nature. Mr. Callan agreed that his paper described laboratory tests but that with time, deteriorating influences acting on field concrete might produce the same results as obtained by accelerated tests in the laboratory.

Simple tests determine coefficients of expansion

A paper presented by D. L. Bloem, assistant director of engineering, National Ready Mixed Concrete Assn., in conjunction with Stanton Walker, director of engineering, National Sand and Gravel Assn., and W. G. Mullen, former Stephen Stepanian Research Fellow of the National Ready Mixed Concrete Assn., dealt with the effect of temperature changes on concrete as influenced by aggregates. Their studies indicated that differences in coefficients of expansion of the ingredients set up stresses which may contribute to disintegration of concrete but that the relationship between thermal properties of concrete ingredients and durability are not yet well defined. The tests furnished good information on coefficients of expansion of concrete made with varied proportions of different aggregates. They indicated that quick changes in temperature are destructive and that concretes having low coefficients are more resistant to such changes than those having high coefficients.

In discussion of the two papers, Prof. C. H. Scholer observed that nature speaks a very complicated language, that the two papers were highly significant, and that we should attempt to find out what nature is trying to convey through the information they present. Further discussion considered the possibility of extrapolating the reasoning of these papers to temperature changes at higher levels in a semi-refractory use for concrete where the thermal diffusivity differs between aggregates and the mortar and may cause great temperature differences between two adjacent pieces. Mr. Callan felt that an extrapolation of this nature would be in order.

Calcium chloride

J. J. Shideler, Bureau of Reclamation, Denver, Colo., presented a paper on the effect of type of cement, age, temperature and mix proportions on the strength of concrete containing calcium chloride, and the effect of calcium chloride on volume change, heat generation, time of set, alkali-aggregate reaction, resistance to abrasion and sulfate attack of concrete. The author covered almost all phases of the use of calcium chloride in concrete.

Discussion following the paper brought up the question of the effect of calcium chloride on the appearance of concrete. Mr. Shideler answered that as far as he knew, calcium chloride may improve the appearance of the surface not in respect to color but in respect to texture.

Further discussion concerned the use of calcium chloride with cold water and cold aggregates and whether or not this had any effect on the concrete. In answering, Mr. Shideler pointed out that they had made concrete mixes and cured them at temperatures from 16 F upward. He said that calcium chloride is effective at any temperature. Data showed that even concrete in a frozen condition containing calcium chloride will gain strength, a thousand pounds or so a year. Tests also showed that even where concrete was maintained at 16 F it had considerable increase in strength.

Conventioners travel far

While naturally the Cincinnati area provided the largest group attending the 48th annual convention 31 other states, District of Columbia, Canada and three other countries were represented. Professor Chuzo Itakura, Hokkaido, Japan, came the greatest distance. Other globe trotters were Dr. K. Hajnal-Konyi, London, England, and Ramiro Parada, Albarico, Venezuela.

Ohio	242	Texas	5
Ill.	76	Wash.	5
N. Y.	37	W. Va.	5
Canada	32	Miss.	4
Ind.	32	Tenn.	4
Pa.	28	Conn.	3
D. C.	27	Fla.	3
Mich.	22	Minn.	3
Mo.	18	Ga.	2
Wis.	15	Kan.	2
Ky.	14	Me.	2
Calif.	12	Del.	1
Va.	12	N. C.	1
Iowa	11	Okla.	1
Mass.	9	Ore.	1
Colo.	7	S. D.	1
Neb.	7	England	1
N. J.	7	Japan	1
		Venezuela	1

Fly ash inhibits cement-aggregate reaction

Chicago fly ash used in quantities up to 20 to 30 percent by weight of the cement plus fly ash will inhibit certain types of cement-aggregate reaction in concrete. This was proved by tests on 3 x 4 x 16-in. beams reported by G. M. Smith in conjunction with C. H. Scholer, assistant professor and professor of applied mechanics, respectively, Kansas State College. The beams were subjected to two different accelerated exposures to determine their cement-aggregate compatibility. These laboratory results correlated very well with field service records.

The use of fly ash aided the workability and finishing characteristics of the concrete and although the early strength was somewhat reduced, strength improved over a period to the point where it was far superior to concrete without fly ash.

Professor Scholer, in answer to a question concerning the effect of water on expansion, believed that it was of little consequence and that the major effects were caused by the character of the aggregate. This opinion was reinforced by Bryant Mather who mentioned a report of tests recently made along this line.

A request for a specification governing fly ash was answered by Bryant Mather. He mentioned that ASTM Committee C-9 has a subcommittee on methods of testing specifications for admixtures and that a tentative specification should be forthcoming. Also mentioned was the fact that various highway departments have already prepared specifications for fly ash.

Bond stress theory not applicable to welded wire fabric

The term "bond" as applied to smooth or deformed steel bars does not define the resistance to slip of welded wire fabric according to Arthur R. Anderson, partner, Concrete Engineering Co., Tacoma, Wash. He demonstrated this in a series of tests on slabs and pull-out specimens. Anchorage better defines the function of the welded transverse wires which provide the resistance to slip. Slip-resisting qualities are proportional to the strength of the welded transverse wires. In the pull-out tests, Dr. Anderson found that slip and transmission of tension beyond the first embedded transverse wire were proportional to the size decrement between the transverse and longitudinal wires. For decrements up to four wire sizes, the first welded intersection provided sufficient anchorage to prevent any transmission of tension for pull-out loads up to 30,000 psi. He concluded that welded wire fabric should be specified on the basis of the size ratio between longitudinal and transverse wires and not conventional bond stress.

DESIGN

Two years of experimentation and investigation promise bright future for lightweight structural concrete. Economy in design aided by tables and graphs. A 305 bars promise greater resistance to cracks. A numerical design procedure simplifies analysis of skewed rigid frames and arches.

Lightweight structural concrete

John A. Murlin, structural engineer, of George L. Dahl, Architects and Engineers, Dallas, Texas, and Cedric Wilson, vice-president and chief engineer, Texas Industries, Inc., Dallas, Texas, discussed the rapid growth of this relatively new material in the past two years. Formerly used for the manufacture of blocks and concrete products only, the authors reported that lightweight aggregates have been proved to be excellent structural materials through study, research and use by a loosely organized Texas group of structural engineers and aggregate producers with a common objective.

Many problems remain to be overcome in design, use of admixtures and standardization, but lightweight concrete is now definitely competitive in the field of structural materials according to the authors.

Balanced design not necessarily most economical

Optimum dimensions almost never, for

C. H. Scholer, Head, Dept. of Applied Mechanics, Kansas State College, recently elected a vice-president of ACI for a 2-year term. Professor Scholer, an ACI member since 1924, has twice served on the Institute Board of Direction

economical design, correspond to balanced design but are dependent upon relative material costs, relative material strength, relative length of beams reinforced for diagonal tension, and relative importance of beam dead weight. Thus concluded Prof. Herbert B. Sawyer, Jr., University of Connecticut, Storrs, who used Whitney's theory and drew his general equation from the following: "If either a small amount of breadth, δb, or a small amount of depth, δd, is added to the concrete cross section, the resultant of the stress block will rise, increasing the moment arm and the moment accordingly. On the other hand, if a small amount of steel, δA_s, is added, the moment will increase since the beam is under-reinforced. When the cross section is so proportioned that the cost of the added concrete equals the cost of the added steel, and the two corresponding increases in moment are also equal, the cross section is most economical for the variables concerned." Professor Sawyer presented these relationships in charts from which may be determined cross-sectional dimensions for maximum economy of materials for a beam or slab with a given loading.

Deformed bars show superior crack resistance

K. Hajnal-Konyi, consulting engineer, London, England, summarized results of tests on beam made with 17 different reinforcing bars, some cold rolled and some with natural yield point. Each beam was tested in 3000 and 6000 psi concrete.

Results indicated that fewer cracks are formed when beams are reinforced with deformed bars but that bar size was more effective than the surface characteristics in this regard.

Verification of Whitney's theory in assessing ultimate bending moment of reinforced concrete, indication that cold worked steel bars allow greater utilization of the ultimate strength of concrete, and that higher allowable bond stresses require higher quality steel, were further conclusions reached in this series of tests.

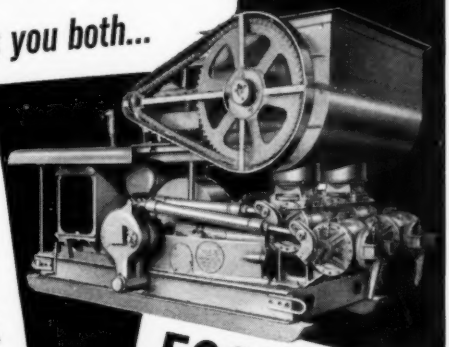
Skewed rigid frames and arches analyzed by numerical procedure

A simple numerical procedure for the analysis of single span skewed rigid frames or arches presented by James P. Michalos, associate professor, Dept. of Civil Engineering, Iowa State College, Ames, gave results identical with those attained by use of classical methods such as Rathbun's.

The paper was based on the normally assumed relationship between the statics and geometry of an element of a structure and a step-by-step procedure for reaching a solution was well illustrated by a series of drawings and charts.

Institute officers, past and present —Seated: M. O. Withey, D. E. Parsons, Harvey Whipple, R. E. Davis, R. B. Young, H. J. Gilkey. Standing: Fred Van Atta, R. F. Blanks, H. F. Gonnerman, H. F. Thomson, A. T. Goldbeck, F. H. Jackson, Stanton Walker

IN TUNNEL LINING
PUMPCRETE® gives you both...



QUALITY

BECAUSE...

- Pumpcrete does not segregate concrete
- Pumpcrete alone can place sidewall concrete in horizontal layers without chuting or dropping of the concrete
- Pumpcrete will handle stiffer mixes
- Pumpcrete reduces loss of slump between the pump and the point of placement
- Pumpcrete will fill overbreaks higher . . . tighter than any other method . . . up to 20 feet
- Pumpcrete assures a continuous flow of concrete to the forms . . . eliminating many cold joints and related imperfections
- Pumpcrete alone assures a tightly lined tunnel without need for grouting

ECONOMY

BECAUSE...

- You can spot Pumpcrete in the most advantageous position . . . above ground . . . inside the tunnel . . . outside tunnel entrance
- With horizontal layer method specified, Pumpcrete is the only mechanical method that can be used . . . and is best for advancing slope, too.
- In pouring arch, Pumpcrete eliminates expensive grouting since it will fill overbreaks higher . . . and tighter than any other method . . . up to 20 feet.
- Pumpcrete *flexibility* offers the best and most economical method of pouring invert.
- Pumpcrete *flexibility* eliminates need for expensive hauling and re-handling equipment . . . reduces labor requirements.

For the complete Pumpcrete story, write **Chain Belt Company,** 4713 West Greenfield Ave. Milwaukee 4, Wis.

CHAIN BELT REX COMPANY ®

CONSTRUCTION MACHINERY

CONSTRUCTION

Vacuum concrete

K. P. Billner, president, Vacuum Concrete Corp., Philadelphia, Pa., reported on a trip through Europe, Africa and South America during which he saw some of the more outstanding examples of new construction using precast and vacuum process methods.

Vacuum processing is particularly applicable to precasting flat sections, such as walls, roofs, or floor panels for which the same forms may be used several times a day. However, it has been easily adopted to other more complex sections, such as pipe or curved roof sections.

Some examples mentioned by Mr. Billner were precast, vacuum processed canal linings in France, precast vacuum processed reinforced and un-reinforced pipe up to 10 ft in diameter in Italy, and in South Africa, precast vacuum process water tanks with a 200,000 gallon capacity.

Electricity used by Japanese for winter curing of concrete

The construction of the American Army Camp Crawford at Makomani in Hokkaido, Japan, provided the opportunity for the civil engineering research members of Hokkaido University, lead by Prof. Chuzo Itakura, to experiment with the use of electrical curing methods for winter concreting. The structures to be built ranged from plain thin concrete sections to mass reinforced sections.

Previously, climatic conditions in conjunction with a lack of adequate heating methods almost prevented any winter concreting. Prof. Itakura, however, reported that the condition is now being overcome through the use of electricity. This method provides a cheap, rather simple setup with greater advantages than normally found in other winter curing methods, and is gradually coming into use. An added feature of this method is that early compressive strength tends to be higher than in conventionally cured concrete. However, at later ages it returns to the norm. Care must be taken in avoiding continued heating of the reinforcing bars, as a high temperature differential between steel and concrete tends to reduce bond stresses.

In general the method requires that a net of electrodes be spread out through the concrete and connected through a transformer to a source of power. Then current is fed into the electrodes at a specified rate under given conditions to insure a gentle temperature rise. Further work and experiment on this method may eventually make winter concreting in Japan an economical and practical procedure.

Use of concrete in residential homes reaches new high in 1951

Residential building required over 3,000,000 sq ft of floor slab in 1951. This large volume of concrete indicates the importance of emphasis on good construction practice in home building. The fact that relatively light loads are normal in this type of building suggests than an addition to a section or a separate section on home building be incorporated in the ACI Building Code. This point was made by C. O. Christenson, associate director, Technical Dept., National Association of Home Builders, in his paper on the use of concrete in residential building.

He also discussed the construction of a housing project at Great Lakes Naval Training Station, Illinois. This project was built on a mass production basis, but nonetheless, good construction practice, adherence to ACI Building Code, and sound design were employed.

A tapered footing was placed first and then the walls were precast in long forms which made it possible to cast all the walls for a building at once. The walls were erected by the tilt-up method, and then were joined by welding together previously embedded plates.

A discussion of two different types of footings for one-story houses, one to be used in mild climates, the other in more rigorous zones, concluded the paper.

Wood shavings insulate new winter concrete

L. H. Tuthill and R. E. Glover, research engineers, Bureau of Reclamation, Denver, in collaboration with C. H. Spencer and W. B. Bierce, construction engineers, Hungry Horse and Fort Sumner Projects of the Bureau reported on a new method of protecting winter concrete.

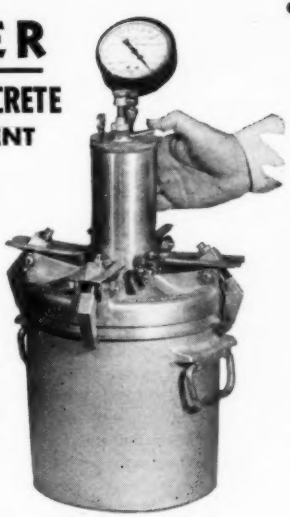
PRESS-UR-METER

FOR TESTING AIR ENTRAINED CONCRETE

COMPACT . FAST . CONVENIENT

GUARANTEED ACCURACY

SAMPLE REMAINS INTACT: Small amount of water used in test permits using same sample for slump and compression tests.



Now in World-Wide Use!

Universal acceptance: U.S. Bur. of Recl., Army, Navy, Public Roads; many St. Highway Depts., Commercial Laboratories, Ready Mix Plants; Major Projects in N. & S. America, Europe and Asia. SPECIFIC GRAVITY and MOISTURE DETERMINATIONS of aggregates quickly made using the NEW CHART now furnished with the PRESS-UR-METER. LET US TELL YOU about this important extra value of the PRESS-UR-METER for testing and designing concrete mixes.

For complete information, write to

CHARLES R. WATTS & CO.

4121 – 6th Ave. N. W. Seattle 7, Wash.

Exclusive sales agents

Formerly protection for new concrete required that it be placed at a temperature of 50 F or above and kept there for three days and protected from freezing for the next 11 days. Recent investigations have shown that concrete with 1 percent calcium chloride if kept above 50 F for three days and above freezing for the next three days will be sufficiently protected against freezing.

The new requirements suggested that old-style heating methods might be eliminated if the heat of hydration in the concrete could be kept from dissipating. The first thought was of insulation that might effectively prevent heat loss to the extent required.

Insulation such as straw had been used to some extent previously but this was the first time it offered serious possibilities. First field tests were made at Hungry Horse Dam using planer shavings inside Sisalkraft paper placed against the form. It was intended that when forms were removed the paper enclosed shavings would be moved up against the concrete to give some additional protection beyond the required time. The tests were successful and amply demonstrated the potentialities of this method. Further tests on canal linings using a somewhat different type insulation were reported equally successful.

Further experiments were performed in the laboratory and curves and formulas evolved which aid in anticipating the insulation requirements in view of expected weather conditions.

Centriline process of pipe lining

A movie produced by the Raymond Concrete Pile Co. was presented by Ken Kirwin of that company. The film discussed the reasons for deterioration of metal pipe lines and the equipment for cleaning and lining them with cement mortar by the Centriline process. The effect of the lining on the carrying capacity of the pipe was considered and brief data given on tests to determine the effect of distortion and of holes in the metal pipe on the efficiency of the lining. Two other methods of lining also were mentioned.

RESEARCH

The ten excellent papers presented at the research session in Cincinnati covered a wide variety of topics.

Accelerated natural weathering tests

W. J. McCoy, Lehigh Portland Cement Co., Coplay, Pa., in his discussion of accelerated natural weathering tests of concrete, stated that selected cements representing principal types were made into 3 x 3 x 15-in. concrete bars and 3 x 6-in. concrete cylinders which were stored outdoors in concrete trays partially immersed in water to a depth of 1.5 in. These specimens, which now have experienced 10 years of exposure, include comparative tests of 4.5 and 6 sacks per cu yd concrete at a slump of 3 in. and also tests of 3, 6, and 0 in. slump concrete at a constant W/C ratio. The sonic method and visual appraisal were employed in most cases to evaluate the results.

Correlation between durability in natural and artificial exposure

T. B. Kennedy, Concrete Research Div., Waterways Experiment Div., Jackson, Miss., discussed the correlation between durability in laboratory freezing and thawing and exposure at Treat Island. In these tests, one hundred fifty 3½ x 4½ x 16-in. beams and one hundred fifty 6 x 6 x 30-in. beams, all of air-entraining concrete, representing combinations of six coarse aggregates with four natural and four manufactured sands, were exposed on the Corps of Engineers exposure rack at half tide elevation at Treat Island, Cobscook Bay, Eastport, Maine, during the late fall of 1948. One hundred fifty 3½ x 4½ x 16-in. companion specimens were tested in the Waterways Experiment Station, Concrete Research Division laboratory automatic freezing and thawing machine. Insufficient time has passed to date to draw definite conclusions, but indications are that for all the aggregate combinations tested, the laboratory exposure, cycle for cycle, is more severe than the Treat Island exposure, but when compared on a "degree cycle" basis the Treat Island exposure in a number of cases appears to be more severe.

Durability studied using low temperature nitrogen sorption

C. M. Hunt, National Bureau of Standards, Washington, D. C., discussed the application of surface measurements by low temperature nitrogen sorption to the study of freezing and thawing. He reported on a preliminary investigation of the measurement of surface of mortar, brick, and stone specimens before and after freezing and thawing. The gas adsorption method, particularly good for detecting so-called internal surface in solid adsorbents, was used to determine whether changes occurred in specimens during freezing and thawing and to correlate initial surface with resistance to freezing and thawing.

High pressure steam curing

W. C. Hansen, Universal Atlas Cement Co., Gary, Ind.; in his report of the activities of ACI Committee 716, High-Pressure Steam Curing, stated that the committee has undertaken a cooperative study to determine the physical properties of high-pressure steam-cured blocks and to develop rapid methods for determining these properties. Each of three companies is furnishing blocks made with sand and gravel, cinders, and expanded clay aggregate for this study. Four laboratories are determining the absorptions, shrinkages, and specific gravities of these blocks by three different methods. All laboratories are testing some of the blocks for compressive strength, and some are making tests for tensile and flexural strengths.

Fatigue tests of plain concrete beams in flexure

C. E. Kesler, University of Illinois, Urbana, spoke on fatigue tests of plain concrete beams in flexure now under way at the University in which it is hoped to determine the effect of rate of load-application, the frequency of loading, and other time effects on the finite life and endurance limit of plain concrete

subjected to repeated loading. Four beam-type testing machines, operating at three different frequencies, are used to test plain concrete beam specimens. A unique method has been developed to determine the percent of ultimate static strength at which a specimen is tested. Sixty specimens of 4000 psi concrete have been tested.

Dynamic, elastic and plastic tests on reinforced concrete beams

J. S. Archer, Massachusetts Institute of Technology, Cambridge, spoke on dynamic tests on reinforced concrete beams in the elastic and plastic range. His tests have shown that concrete beams reinforced with structural grade steel or with rail steel will absorb more energy when loaded impulsively than when loaded statically to the same total deflection in the static range. Some differences in the behavior of the two types of steel was, however, obtained. Prestressed reinforced concrete beams also absorb more energy under dynamic loading than under static loading, but the gain is less than for structural grade steel. He reported that a reinforced concrete beam that fails in shear is inefficient and unreliable and that its capacity for absorbing energy is very small because of the small ultimate deflection tolerated by it.

Stress gage measures distribution of compressive stress

Douglas McHenry, U. S. Bureau of Reclamation, Denver, reported on stress gage measurements of distribution of compression in a reinforced concrete beam. He noted that in 1939, Roy W. Carlson devised a meter for the direct measurement of compressive stress in the interior of concrete, and that the Bureau of Reclamation laboratories have since constructed similar but smaller meters and have used them to measure the distribution of compression in the region above the neutral axis of a reinforced concrete beam. The stress measurements were supplemented by measurements of strain on the surface of the concrete and on the reinforcement. The test beam was 32 in. deep, 13 in. wide, and 26 ft long; it was heavily reinforced (4.3 percent longitudinal steel) to assure a compression failure in the concrete. The strain gages between the neutral axis and the top fiber indicated a linear distribution of strain up to the failure load. Corresponding stress gages showed a nearly linear distribution of compression at the lower loads with pronounced curvature developing as failure was approached. This curvature appears to explain satisfactorily why the beam supported a load 36 percent greater than the maximum load computed on the basis of linear distribution of stress.

Ordinary reinforced and prestressed reinforced concrete beams compared

A. C. Loewer, Jr., Lehigh University, Bethlehem, Pa., presented a comparison between ordinary reinforced and prestressed reinforced concrete beams. The purpose of this research was to compare the physical properties of ordinary reinforced, pre-tensioned prestressed and post-tensioned prestressed concrete beams. An 8 in. x 12 in. x 12 ft beam of each of the above types was tested in the Fritz Engineering Laboratory of Lehigh University. Concentrated loads were applied to the third points of the 11-ft test span and measurements of deflection, crack patterns, and strain in the concrete were recorded. SR-4 strain gages attached to the surface of the concrete were used to indicate strain, which was later interpreted as stress. The tests resulted in much informative data and were useful in the planning of a program now under way on large prestressed concrete beams.

In addition to the above mentioned papers, A. G. Timms and G. L. Kalousek also spoke, but information on their projects is not available for public release at this time.

Harvey Whipple acknowledges Institute thanks for his more than 32 years' service

Photo courtesy Rock Products

Meet your

By HARVEY

After looking forward somewhat apprehensively to this, to me, momentous occasion, with my fingers too tightly crossed, I am much reassured in seeing here so many friendly and familiar faces of those without whose help I could not have worked so enthusiastically and at relative peace with myself for a long time.

May I break through what has been said so genially by my good friend, Herbert Gilkey, about a long-haired but perhaps only slightly Bohemian past, through a period of change, many years ago, to somewhat more formal neckwear, in keeping, I thought, with the triple-deck equations of my ACI associates.

And now with my thought on Fred Van Atta who takes over very shortly as my successor, I wish him well and as much fun as I have had with, in general, the same kind of people.

Most of all, after mentioning Fred, it is my special pleasure to ask all of you to keep in mind others of my fellow "staffers." They work mostly in the background. You ACI members depend upon that work. I believe they can and do *help* you get more from the work of all of us. Certainly we are not heavily staffed. A dozen of us try to do it all, including ten JOURNALS a year with some highly pertinent *applications of knowledge in practice* – – – or whatever our Board directs.

At this table is Mrs. Ethel Wilson who has been with us almost as long as I—but I shall not embarrass her by being more specific than to assure you she was very young when she started with considerable zing, which she retains, even after going back to school evenings, time and again, to learn better the quirks of dollars and cents record-keeping to satisfy you and our professional auditors.

Bill Maples isn't here. Some must "keep home fires burning" and I was happy to wire him congratulations on his advance at this week's meeting to "managing editor," almost coincident with Fred Van Atta's assumption of more general responsibility to and for ACI.

Bernard Cronk, our assistant editor, is in our midst—his first ACI convention—cramming a maximum of valid impressions of what ACI is all about.

At Detroit headquarters feeding the home fires, and I'd like you to feel you know them, are:

*From remarks at the Harvey Whipple Luncheon, Netherland Plaza Hotel, Cincinnati, Ohio, Feb. 28, 1952.

ACI staff

WHIPPLE*

Mrs. Isabelle Davis, manager of publication sales, sits close to the entrance where callers often come personally seeking more concrete knowledge, or by mail, flood her desk with their orders and inquiries.

Mrs. Dorothy R. Hall, whom many of you know indirectly only through correspondence, is editorial secretary and alert to many office details with overlaps of departmental responsibility. She sorts the mail, implements the work of the Institute from the editorial slant and aims to keep new information where we can find it.

Mrs. Betty Haltom, in charge of membership, has been with us since December 1947, first as Betty Morris. She has very recently married; her husband is in the U. S. Army in Japan.

Barbara Buresh has inherited much editorial correspondence in her three and one-half years with ACI, and since December 1951 has been our news editor.

Mrs. Evelyn Horowitz is assistant to Mrs. Wilson and has completed her first year with ACI.

Margaret Miller has been with us a year and assists Mrs. Davis in invoicing and mailing several thousand publications orders a year.

Mrs. Katherine L. Mullen has just completed a three months' period with ACI as a pinch-hitter closing staff gaps in several departments.

I've been having a whale of a time and perhaps I have enjoyed prickly incentives of zigzagging so often up and down—always in some crisis in shifts of membership weather. Wars and rumors of wars have interrupted things we were trying to get done. A few of our people have stuck by us and helped get them done and I should like to have you feel when you write to the Institute for help on any one of hundreds of problems in our field that you are writing to these people whom I named. The American Concrete Institute is, after all, "folks," folks who direct and folks who execute the directives as we are trying hard to do.

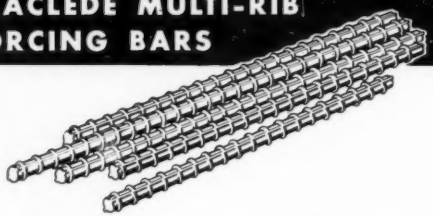
We made an effort to learn the leanings of our member interest and there appear to be about 120 different *classifications*. I have come to the conclusion, and trust Mr. Van Atta shares it fully, that we asked the wrong questions or asked them in the wrong way. Words indeed are tricky tools to work. We should ask questions differently and so consolidate our thoughts. The Institute is above all else people; going many different paths to a *few* major goals.

It has been, still is, great fun. Thank you very much.

BETTER STRUCTURES

BEGIN WITH THE BALANCED STRENGTH OF LACLEDE MULTI-RIB REINFORCING BARS

Foundation of Central Catholic High School, East St. Louis, Illinois. Saunders and Johnson, Associated Architects Wm. H. & Nelson Cunliff Co.—Contractors



Designed to new A. S. T. M. specification A-305, Laclede Round Multi-Rib Reinforcing Bars actually provide increased anchorage . . . offer opportunities for appreciable savings in steel and concrete through the elimination of hooks and shortening of embedment lengths.

Specify Laclede Multi-Rib Reinforcing Bars by number . . . in standard sizes of the new A.S.T.M. specification A-305 Table I.

For the Best in Reinforcing Steel . . . Specify Laclede.

LACLEDE

Producers of Steel for Industry and Construction

LACLEDE STEEL COMPANY

St. Louis, Mo.

PANEL SESSION

Chairman Gilkey and his group of experts, M. D. Catton, R. E. Copeland, R. W. Gilmore, Edward C. Harding, Eivind Hognestad, Charles T. Kennedy, F. N. Menefee, Nolan D. Mitchell, J. A. Murlin, J. P. Thompson, and C. E. Wuerpel, made the panel session one of the convention highlights again this year. A wide variety of questions were asked and answered and everyone there felt they had received answers to at least one or two questions that had been bothering them for some time.

How does one build concrete that doesn't crack?

Professor Gilkey answered this question immediately after asking it. "One doesn't," he said. Miles D. Catton added that the question is not how to avoid cracking but, "how to design our structures to control the cracking. The architectural engineer is, of course, very conscious of the need for controlled cracks and analysis of stresses in his members to properly locate them. The concrete masonry man likewise is aware of the need of control of cracks through proper location of control joints. The concrete pavement man had worked on the problem for years. Like the pendulum we swing from one extreme to the other, no control of the cracks at all in the twenties, swinging over to putting about a ½ in. expansion joint every 30 ft, and now today we are swinging back to the spot where we are pretty sure that a contraction joint every 15 to 20 ft will do a good job. Obviously the design and control of the mix enters into the design of crack control, and the cement chemist is as interested in that question as anyone else." Mr. Murlin added that as Mr. Catton said, concrete is going to crack. The main thing to do is go ahead and let it crack, "but make it crack where you want it to crack, which is of course the science of control joints in structures."

M. O. Withey advocated a look into the constitution of cement in an answer to the cracking question.

Mr. Kennedy expressed a converse opinion. "I think we overemphasize the necessity for expansion, or perhaps more properly speaking, contraction joints in reinforced concrete construction. They are not as necessary nor serious as text books will tell you. I have in mind a reinforced concrete crane way, consisting of reinforced concrete girders and columns, 500 ft long without any expansion joint. A thorough inspection of that building some years after it was built revealed no significant cracks whatsoever. There were a few small hairline surface cracks. I believe too much money has been spent in industrial building design to provide expansion joints, most of which don't work."

R. C. Sandberg reinforced Mr. Kennedy's opinion, mentioning several buildings that he has seen which have many cracks in them but otherwise show no sign of deterioration or failure. "Particularly in mind is a structure not far from where I live, an armory building in which the control joints apparently were studied quite carefully. But somehow these cracks seem to have developed in some place other than the location intended. Apparently nature didn't like the locations that had been assigned, so the cracks occurred elsewhere, sometimes only a few inches away from the control joint."

Douglas E. Parsons, with regard to hydraulic structures, said, "it pays to use small, well-deformed bars closely spaced, and in that way be assured that the cracks will not be too wide to be tolerable. Under very high heads one may have cracks of 0.0002 in. wide without any serious difficulty."

Is there such a thing as form design or are forms still left to judgment and rule of thumb?

Professor Hognestad mentioned several books which included form design formulas, the best being one put out by the Danish firm of Christiani and Nielsen which considers four variables; consistency of concrete, placing speed, setting time of concrete, and the shape of the forms.

To actually determine the pressure on forms is difficult. "Pressure cells have been used and deformations of the forms have been used, both of which are unreliable." He said that the best method to his thinking is the electric strain gauge fastened to either

steel wires or flat steel strips placed in the forms. Mr. Harding said there is much involved in the design of forms, not so much in the structural design, but in the economical use and reuse particularly, on a structure of any size. "On a multiple story building, form design, and the routing through the various reuses, is very definitely a subject that can be dealt with in the office."

The inadequate strength of forms has produced many sad experiences according to President H. F. Thomson. "When somebody along the line, either through lack of information or knowledge of design, cuts the form cost they may get a form that gives way. Such failures often cost many times what would have been involved originally if a little stronger construction had been used. I think there is something very definitely needed in the way of further information as to pressure that occurs in the forms, particularly in high foundation walls."

R. C. Sandberg had experienced good success with steel forms in the lower levels of buildings. However, in cases where the structural concrete was exposed on the outside of the building and was therefore architectural concrete the success was not so good. He found that steel forms did not lock sufficiently tight at the corners and some bleeding occurred which spoiled the looks of the architectural concrete. He felt that steel forms give an excellent surface but until this corner problem is licked it will be difficult to use them on exposed surfaces.

S. J. Warberg said that in many cases the cost of forms pretty nearly determined the cost of the work. Therefore, forms should be designed for as much reuse as possible. Robert C. Johnson verified this and added that if designers took into consideration lumber sizes a considerable cut in the cost of forming could be made. For example, make 12 in. beams 11⅝ in.; design the span between beams so that 12, 14 or 16 ft 2 x 4's could be used by sawing in half rather than calling for a 6½ ft span which requires wasting a foot of lumber when cut from a 14 ft length. Another point brought out by Mr. Johnson, "I would like to mention that failure of wall forms or any other type of structure isn't too important to me as a financial loss. The major worry is the failure of a floor slab while it is being poured, because my carpenters are under it continuously and if that floor slab goes down I am going to kill a man. That is why it is very important that research be done in the spacing of shores or the size of the girts that you can use, particularly a study of your cross frame bearing."

A 305 bars have relatively deep lugs. How does the ratio of tensile strength to weight of bar compare to that of plain bars of comparable grade steel?

J. P. Thompson, Secretary of committee 318, said "I think this was answered in one of our committee meetings here this week. The first thought is that considerable area of bar is lost. There are no limitations in the ASTM specifications for bars on that point. As a matter of fact, it was mentioned in this discussion that some tests had been made with ⅝ in. bars where they machined off deformations and found there was a loss of section of about 9 percent in those particular bars.

"Some bars are supposed to have approximately equal cross sections throughout their length. Whether or not this is true or what effect it has is still beside the point, because as someone said you do not buy a cross sectional area of bar any more. You buy a strength of bar, that is, if you buy a No. 5 bar, you do not get a bar which has an area of 0.31 sq in. at every point. What you buy is a bar that weighs as much as a ⅝-in. round bar of the same length. Consequently, when ordering intermediate bars with a minimum yield point of 40,000, you get a bar which has a yield point of 40,000 times the theoretical area."

Is radiant heating for concrete floors and ceilings now fully practicable and reliable?

R. E. Copeland answered "I have been in a number of homes and a few buildings that were heated with radiant heating, both hot water type or warm air type, and based on what I have been told by the people who live in these homes or buildings, and undoubtedly they are the best judge, I would say that if the system is properly designed and is installed by a competent heating man, that these systems are just as

reliable as the older traditional system. I do think, however, that they are not as fool proof from the standpoint of installation as the older systems. I think that they require more competent engineering than the conventional systems."

Mr. Mitchell added, "There are drawbacks, of course, in a system of this kind. In those in which you have the heating elements, usually pipes, filled with hot water which is circulated by a pump, the response to changes of temperature is rather slow. However, there has been developed a type of control that is actuated not by the temperature within the house, but the temperature outside, so that when there is a drop in the outdoor temperature, there is a prompt response so that there will not be too much delay in getting the necessary heat into the living quarters. And, on the contrary, if there is a rise of outdoor temperature, then the outdoor control anticipates a reduction of heat demand.

"Mr. Sandberg mentioned that there is evidence to show that radiant heating installed in the ceiling may give better results and be more desirable than that installed in the floor."*

What are the effects of placing methods, tamping, vibration, and pumping, on the entrained air in concrete?

Panel member C. E. Wuerpel answered, "I think that is a rather large question, and yet it might be answered in relatively few words. The air entrained in concrete does not affect the vibration of the mixture more than does any other constituent of the mix. When we acknowledge fully that the entrained air is a separate and distinct entity in the mixture, and when the mixture is properly designed to have a balanced condition without excess of any one of the ingredients, no trouble should be experienced with vibration or in pumpcreting. During the time I was with the Corps of Engineers, serious difficulties were had with concreting on several projects when air-entraining concrete was used. In each instance the basic trouble was not with air entrainment, but was due to the mixture not being properly designed for use with air entrainment. When proper adjustments were made in the sand content, the concrete pumped very well. When mixtures are over-sanded, there is a concomitant tendency toward excessive air entrainment. If you build up a high air content, the mixture develops an elasticity against which the piston works ineffectively.

With well-designed air-entraining concrete mixtures, concrete has been pumped efficiently well over a thousand feet.

Vibration has a characteristic with which we are generally familiar. With an excessively harsh mixture in a relatively confined area, it is possible actually to vibrate some of the excess stones out of the mixture. If there is an excess of sand, water, or air in the mixture, that component works to the top in a similar manner. Thus, vibration induces segregation in poorly designed mixtures with or without air entrainment.

Exhaustive tests indicate that with thorough, but not excessively prolonged, vibration of a well-designed mixture, the total air entrainment may be reduced by about 1 percent to 1½ percent. The air that is excluded by such vibration is usually that which was incidentally entrapped in the form of relatively large bubbles. There is no tendency to vibrate out appreciable quantities of the small bubbles of purposefully entrained air in concrete mixtures unless *very* prolonged vibration is permitted.

As for tamping and spading, the presence of air gives concrete mixtures a rubbery consistency. Therefore, tamping may be made a little more difficult until the workmen learn how to handle the mixture with tampers and spades."

Thomas B. Kennedy added, "We have done a little work since Mr. Wuerpel was with us on the effect of vibration on air content of concrete, and as he says, the first air to come out is the air that is incidentally entrapped in the concrete; this may be as much as 1½ percent. But we have found that with continued vibration in excess of what is needed for placement and effective consolidation, the entrained air in the concrete can be exhausted. We found that as much as 3 percent of air can be lost with a very husky, fairly high frequency vibrator, in 30 seconds in excess of the time needed for consolidation.

M. F. MacNaughton said that in Canada some gravel deposits used for aggregates are

*Vanderweil, R. G., Disc. of a paper by John R. Nichols, "Radiant Heating by Reinforced Concrete," ACI Journal, Dec. 1946 Part 2, p. 516-3, *Proc.* Vol. 42.

glacial in origin and that there have been in the course of the last three or four years a dozen cases where they have accidentally entrained not 1½ or 2 percent air but as high as eight or nine percent in the mixture. This may cause the concrete to change rapidly from a good workable mix to a harsh unworkable mix due to the instability of the accidental foam, and this of course prevents effective pumping. This difficulty has been overcome he said, by purposely entraining an excess of air and at the same time adding an anti-foaming agent to eliminate the accidentally entrained air.

"To leave a residual of 4 or 5 percent of properly entrained air in the mixture, concrete could be delivered from the mixer in dumpcrete trucks, hauled 3 miles, put through a pumpcrete machine, pumped a thousand feet and not lose more than 2 percent by the time it was finally placed in the forms."

What are the uses and abuses of shotcrete in patching and in structures?

R. W. Gilmore answered for the panel, "We find that in lots of places shotcrete is an excellent method of repair and we have discovered as we went along that in many cases it wasn't the right method.

"Usually if you have a good enough piece of concrete to start with, you can get a very durable job with shotcrete. I don't believe shotcrete works well on other than concrete, except on stone. On a stone base shotcrete repairs are good; on brick we have had some very bad results. But a lot of people have had good results with shotcrete, and it may be that our processes aren't quite right."

"I would like to supplement Mr. Gilmore's remarks," said Mr. Harding, "by the observation, here is one phase of restoration work where I think the human equation enters into it very, very largely. A lot of the abuses of shotcrete can probably be attributed to the human element, not only from the actual use of it, itself, but from the decision to use it. Shotcrete has a definite place in restoration work; in other places it is a detriment. And once the decision to use it is made, the human equation enters into it to a considerable extent."

Mr. Mitchell reported that shotcrete is not always successful as a fire-resistant material. "I think that you will find the ordinary shotcrete of sand and cement will give disappointing results when exposed to fire, even after long drying periods, because of the very density of the material. Therefore, I would caution against the use of it unless you know more about it than we have been able to learn in our fire testing experience. However, by including some sawdust in the mixture we have greatly improved its fireproofing characteristics."

COMMITTEE MEETINGS

Twenty committees and subcommittees held meetings during the convention. It was an excellent example of the importance ACI members attach to their responsibilities in committee work. Reports of some committees were not received in time for publication in this issue.

Committee 115—Research (Executive Group)

S. J. Chamberlin called the meeting to order with members D. E. Parsons, R. E. Davis, H. F. Gonnerman, C. P. Siess and G. W. Washa present. Dissemination of the information in the annual compilation "Research Projects in Plain and Reinforced Concrete" to all important interested foreign groups were approved. Further, ACI Committee 323, Prestressed Concrete, was given permission to use the information in the report to further their objectives.

It was recommended that room be provided in the annual questionnaire for listing the active and sponsoring agency of the project together in order that duplication of projects be avoided. This could possibly happen with the present setup of having the sponsoring agency in one part and the active agency in another.

Committee 210—Resistance to Erosion in Hydraulic Structures

Committee chairman W. H. Price and members R. R. Clark and R. B. Young along with eight visitors met to discuss a report on erosion by cavitation and chemical action. Suggestions and changes will be incorporated in a revised report to be resubmitted to the committee for approval and publication.

Committee 213—Lightweight Aggregates

Chairman George W. Washa welcomed H. T. Williams, J. J. Brouk, C. C. Carlson and F. H. Jackson (for A. G. Timms) to a committee meeting held February 25. It was decided that a tabular form would be used for presenting information on lightweight aggregate. The tables will be brought up to date and information on manufactured slate aggregates will be added. The revised table will be the basis of the committee's report to ACI.

Committee 314—Rigid Frame Bridges

Chairman D. H. Pletta welcomed members T. Y. Lin, Douglas McHenry, James P. Michalos, A. L. Parme and guests to the convention meeting of Committee 314.

Analytical work and exploratory experimental work already accomplished were discussed first. The remainder of the meeting dealt mainly with the proposed model tests of skewed rigid frame bridges. Type and scale of model, or models, and instrumentation were discussed. Several proposals are under consideration, and it is expected that final decisions regarding these matters will be reached soon, making it possible for the tests to get under way this year.

Committee 315—Detailing Manual

The following committee members were present: R. C. Reese, chairman, John F. Seifried, Raymond Archibald, Frank Kerekes, E. E. Rippstein, C. J. Kuhn and A. M. Reese. Several visitors also attended.

The principal business was the determination of how to use the "Manual of Standard Practice for Detailing Highway Structures," whether in conjunction with the building manual or separately. Estimates of the possible savings of money that could be anticipated by publishing them as one book are to be investigated. Raymond Archibald was appointed as a liaison between AASHO and Committee 315.

Also discussed was the writing and distribution of a teaching syllabus which would explain the use of the Manual as a part of student training in designing and detailing structures.

Subcommittee 7 (Committee 318)—Shear and Diagonal Tension, Bond and Anchorage

The following members were present: R. C. Reese, chairman, C. A. Willson, Fred F. McMinn and L. C. Urquhart. Several guests also attended.

The meeting was devoted to a discussion of diagonal tension, especially in the light of the tests completed by Arthur Clark at the Bureau of Standards, Hognestad at the University of Illinois, and some tests at the University of Toledo.

Subcommittee 8 (Committee 318)—Flat slabs

The meeting of Subcommittee 8 on Flat Slabs was attended by Joseph Di Stasio, chairman, L. H. Corning, W. C. E. Becker and F. B. Brown.

The meeting centered on a discussion of the proposed revisions to the Flat Slab Section of the ACI Code, using a tentative code which was prepared under the direction of the chairman and distributed to the members for their comments and suggestions.

Subcommittee 10 (Committee 318)—Footings

Present at this meeting were Chairman N. M. Newmark, members A. E. Cummings, F. B. Brown, C. A. Willson, F. F. McMinn, W. C. E. Becker and several visitors. Professor Kerekes outlined his plans for revision of the Building Code and asked that Subcommittee 10 prepare a revision of Chapter 12 (Footings) ACI 318-51 within the next year. Discussion followed concerning the sections to be revised and specific persons were chosen to prepare tentative specifications by August 1, 1952.

Pausing during lunch are H. J. Gilkey, A. T. Goldbeck, H. F. Thomson, R. E. Davis, R. B. Young and D. E. Parsons

Retiring Secretary-Treasurer Harvey Whipple and Fred Van Atta, newly named as Acting Secretary-Treasurer of ACI

Photo courtesy Rock Products

Committee 323—Prestressed Concrete

Joint ACI-ASCE Committee 323 met with A. E. Cummings, chairman, and members A. Amirikian, Curzon Dobell, W. O. Everling, Myle J. Holley, Jr., Jack R. Janney, N. M. Newmark, C. P. Siess, Howard Simpson, C. C. Zollman and guests present.

A discussion of the tentative specifications for prestressed concrete notation took place and several small changes were made. They were approved as corrected and are ready for publication. The definitions for the same subject are now being put into a second draft and it is hoped that they will be approved in time to be published with the notations.

Negotiations have been completed and approval granted by appropriate ACI and ASCE authorities to make Committee 323 a joint committee. The ASCE representatives named to the joint committee are Raymond Archibald, Myle J. Holley, Jr., Howard F. Peckworth and R. F. Wittenmyer. Plans for the Centennial of Engineering program were also discussed.

Committee 324—Precast Reinforced Concrete Structures

Present were chairman A. Amirikian, members B. G. Anderson, K. P. Billner, R. W. Kluge, N. D. Mitchell, A. E. Cummings, L. H. Corning and guests.

The preliminary draft of Proposed Minimum Requirements for Precast Concrete Construction was reviewed. Among the items covered in the three-hour discussion were: cement specifications, size and type of aggregate, maximum bar spacing, minimum strength requirements and types of concrete and steel, and use of admixtures in the design and construction of precast elements.

Committee 326—Shear and Diagonal Tension

Ten members and 14 visitors heard a report on a contract made by Reinforced Concrete Research Council with the University of Illinois, previously recommended by Committee 326, to test shearing strength of a reinforced concrete slab under a centrally located concentrated load.

A discussion of the proposed tests of shear strength of concrete members without shear reinforcement followed. Several proposals were made for changes in the test program. These suggestions will be brought before the committee by letter ballot and final recommendations reported.

Committee 604—Recommended Practice for Winter Concreting Methods

All members of the committee were either present or represented by proxy. Chairman M. F. MacNaughton lead the discussion concerning possible revision of the recommended practice which was adopted in 1948.

All members of the present committee, which includes former chairman R. W. Spencer and a substantial number of the members of the 1948 committee, were of the opinion that the greatest need for revision of the existing recommendations lay in the direction of providing more specific information for the benefit of the small job concrete producer.

Committee 611—Inspection of Concrete

Present were chairman J. W. Kelly and members F. H. Jackson, William T. Neelands, R. W. Spencer, I. L. Tyler, R. B. Young and J. N. Mustard (for Wilfred Schnarr) and guests.

The principal subject of discussion was the revision of the *Manual of Concrete Inspection*. A number of valuable suggestions were offered for incorporation in the revised text, particularly with regard to proper handling of test specimens on the job, overloading of truck mixers and use of excessively wet consistencies.

The group also discussed ways and means of bringing to the attention of the construction industry, particularly the engineers and architects of buildings, the desirability and advantages of proper inspection. It was felt that the employer should select qualified men, pay them adequately, train them well and encourage continued study, supply them

President-elect Goldbeck with Harvey Whipple and retiring President Thomson

Photo courtesy Rock Products

with appropriate equipment and technical references, define their responsibility and authority clearly and back them up on the job. Papers and discussions in publications outside the Institute should be encouraged.

Committee 613—Recommended Practice for Proportioning Concrete Mixes

Present were W. H. Price, chairman, and members C. L. Davis, A. T. Goldbeck, F. Hubbard, F. H. Jackson, H. L. Kennedy, T. B. Kennedy, I. L. Tyler, Stanton Walker, T. G. Cleadening (for J. C. Ross) and visitors.

A proposed draft of a revised committee report was discussed and approved subject to editorial and minor changes in tables. It is expected that the report will be ready for publication within the year.

Committee 711—Precast Floor Systems for Houses

Members present were F. N. Menefee, chairman, Stephen J. Chamberlin, secretary, Fred C. Bannon, R. E. Copeland, Elmer I. Fiesenheiser, John B. Henry (for Gayle Price), George E. Large, Leo M. Legatski, Nolan D. Mitchell and Robert Andrew Rossi.

Bar spacing and size of aggregate specifications in proposed revision of ACI Standard, "Minimum Standard Requirements for Precast Concrete Floor Units," were discussed.

Committee 716—High Pressure Steam Curing

W. C. Hansen, chairman, E. T. Andrews, R. L. Barbehenn, C. C. Carlson, H. W. Easterly, Jr., M.W. Ferguson, C. C. Fishburn, S. B. Helms, G. L. Kalousek, J. K. Seldon and J. J. Shideler were present along with some visitors. Data from the first set of blocks tested were discussed and the action taken and to be taken in conjunction with further tests was also considered.

Orchids to the local committee!

E. C. Harding, general chairman of the local committee, and his able assistants deserve congratulations from ACI for the unexpectedly large convention turnout.

Unlimited effort, widespread advertising and promotion brought more than 650 ACI members and friends to the 48th annual convention in Cincinnati; 242 were from Ohio alone. The committee members who aided Mr. Harding in his efforts were Julian B. Carson, publicity, Fred F. McMinn, inspection trip, H. K. Justice, student and engineering club liaison, Hunter W. Hanly, advisor on technical program, John V. Maescher, finance, L. W. Hixson, arrangements and entertainment, Henry D. Loring and C. O. Christy.

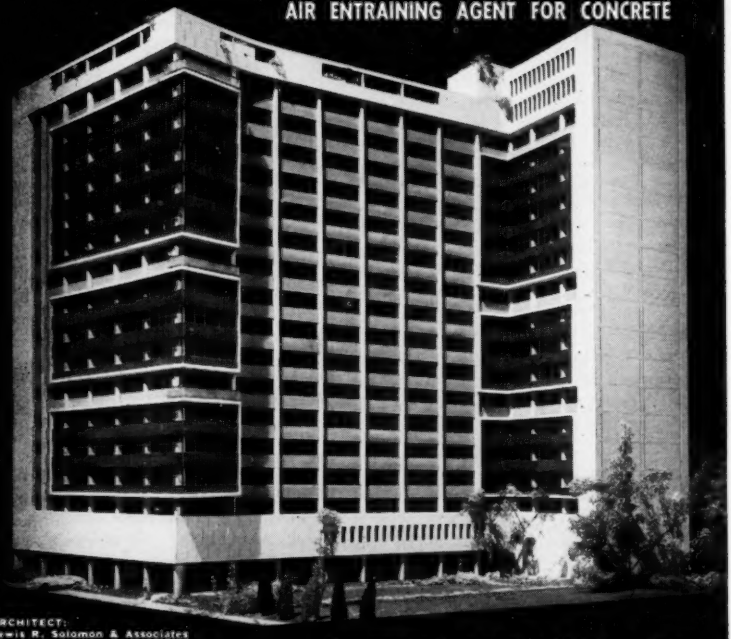
John V. Maescher who gave the welcoming address on behalf of the local committee at the awards luncheon Wednesday noon, spoke from a "non-technical standpoint" and gave a splendid description of the civic organizations of Cincinnati. He also read welcoming letters from Mayor Carl W. Rich and City Manager M. R. Kellogg. Cordial invitations were extended to visit and enjoy the cultural and scientific points of interest in Cincinnati.

The local committee also planned an inspection trip held Friday after the convention. Several members and friends made the trip and enjoyed it thoroughly. The trip included the Millcreek Barrier Dam, Flood Walls in Newport, Ky., Richter Ready Mix Concrete Plant, U. S. Public Health Laboratories Building, the Mariemont Laboratories of the Corps of Engineers and general sight-seeing.

Finished Faster... Better with

DAREX **AEA**

AIR ENTRAINING AGENT FOR CONCRETE



ARCHITECT:
Lewis P. Solomon & Associates
STRUCTURAL ENGINEER:
Alfred Benesch & Associates
CONTRACTOR:
Lloyds Builders, Inc.

New apartment building at 3410 Lake Shore Drive, Chicago. 11,000 cubic yards of Darex AEA concrete. Scheduled for completion October, 1951.

On job after job these facts have been proved: Darex AEA concrete places easier and faster, finishes better, has better surface texture, is more durable when exposed to freezing and thawing.

Ready-mix concrete plants everywhere can furnish concrete that offers you these same advantages . . . concrete made with Darex AEA. Ask your supplier, or write to us for full details.

Cement Products Division

DEWEY & ALMY Chemical Company

CAMBRIDGE 40, MASS., CHICAGO 38, MONTREAL 32

Who's Who This Month

Harry F. Thomson

Retiring ACI President Harry F. Thomson's address at the general luncheon of the 48th annual convention in Cincinnati, "Exciting Adventures," appears on p. 609.

Harry F. Thomson, connected with the concrete industry for many years, is so well known to ACI members that little introduction is necessary.

Johannes Andersen and Poul Nerenst

"Wave Velocity in Concrete," p. 613, is contributed jointly by Johannes Andersen and Poul Nerenst, The Danish National Institute of Building Research, Copenhagen, Denmark.

Mr. Andersen received his MS degree in chemistry from the Technical University of Copenhagen in 1922. From 1922-1928 he was employed by the Danish National Geological Research Institute as chief chemical engineer. During that period, apart from being engaged on chemical analysis of a geological-mineralogical nature, he also participated in geological investigations in the field.

From 1928-1938 Mr. Andersen was assistant lecturer in civil engineering at the laboratory for mortar, glass and ceramics of the Technical University of Copenhagen.

In 1938 Mr. Andersen was appointed chief engineer at the Danish State Testing Laboratory, Section B, Investigation of Non-Metallic Building Materials. He has done research work with these materials, particularly mechanical and physical testings as well as chemical analysis including investigations of relations between strength, elasticity and sound velocity of concrete.

Poul Nerenst received his MS in civil engineering from the Technical University of Copenhagen in 1942. From 1942-1945 he was engaged in designing and supervising the construction of express highways and bridges at the Technical Division, Dept. of Public Works. From 1945-1948 he was chief supervisor at the enlarging of the Copenhagen airport, which is now one of the largest and most modern civil airports in Europe. In this period he studied airport construction in Holland, Belgium and France.

He has been engaged in research at the Danish State Testing Laboratory and in the field on problems relating to winter concreting methods. The application of the condenser chronograph was part of this program.

From November 1951 to February 1952, Mr. Nerenst visited many laboratories and ACI members to study the latest development in concrete technology.

He is secretary of the Section of Concrete and Reinforced Concrete of the Danish Institution of Civil Engineers and a member of an ISO committee on hydraulic binders.

T. V. D. Woodford

T. V. D. Woodford, author of "Slip Forms for Concrete Canal Lining," p. 637, is a materials engineer, USBR, Denver, Colo.

A civil engineering graduate of the University of Kentucky in 1928, Mr. Woodford received his concrete indoctrination in 1930 as a proportioning plant inspector for the Iowa State Highway Commission.

In 1933 he joined the laboratory staff of the USBR where for nine years his activities centered around the physical properties of cements and pozzolans and their behavior in concrete.

During World War II he spent one year as concrete control engineer for McCloskey and Co. in the construction of concrete ships near Houston, Texas.

Returning to the USBR laboratory after the war, his principal function was the coordination of laboratory and field experimentation to reduce the cost of irrigation in canal linings through equipment development, new material adaptions and construction methods. He has been an ACI member since 1940.

I. E. Morris

I. E. Morris, consulting engineer, Atlanta, Ga., tells about "Short Cuts in the Design of Continuous Structures," p. 645.

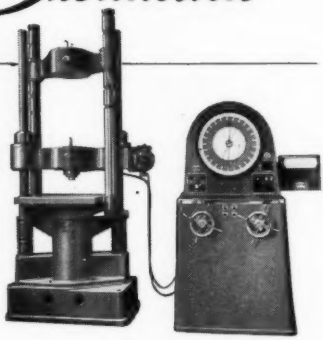
Mr. Morris is a member of ACI Committee 318, Standard Building Code, and is the author of two other JOURNAL papers.

Continued on p. 32

BALDWIN
BLH
LIMA-HAMILTON

THE SEAL OF *Distinction*

BALDWIN-TATE EMERY UNIVERSAL TESTING MACHINES



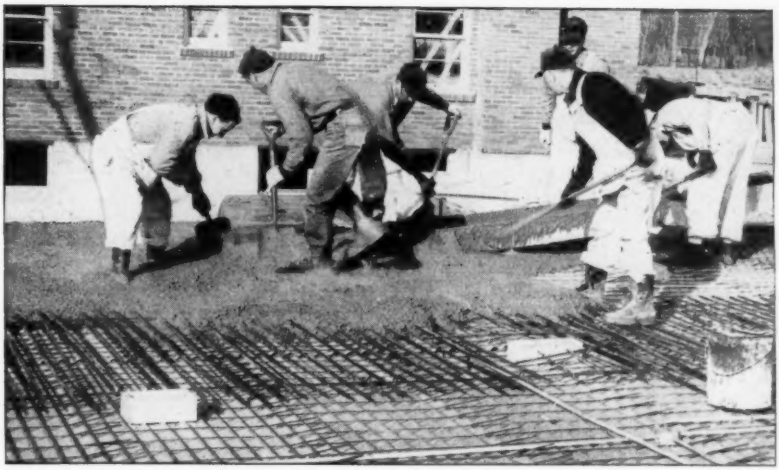
The change in corporate name does not affect the quality, the enduring precision, of the unusual advanced features offered by these world-famous machines.

Tate Emery weighing system is entirely separate from the loading system. Servo drive provides an outside source of power, overcomes drag, provides excess energy to operate auto-controls. The multi-range system permits selection of range during test. 200 to one range or greater . . . accuracy of ½% guaranteed down to 20% of each range . . . A.S.T.M. accuracy guaranteed down to 10% of each range. Sensitivity has been demonstrated at 1 lb. in a million. Zero essentially positive. Negligible hysteresis, creep or temperature errors. Complete calibration and maintenance service. Installation by a qualified Baldwin field engineer.

BALDWIN-LIMA-HAMILTON
TESTING HEADQUARTERS

Eddystone Division, BALDWIN-LIMA-HAMILTON CORPORATION, Philadelphia 42, Pa.
In Canada: Peacock Bros., Ltd., Montreal, Quebec

REINFORCED CONCRETE FLOORS *WITHOUT FORMS*



Yes, Cofar successfully eliminates the costliest element of reinforced concrete construction—wood forms. In one manufactured product Cofar serves as complete positive reinforcement, temperature reinforcement, and form combined, without extra steel and without reduction of design stresses. Cofar is fabricated from extremely high-strength cold-rolled deep-corrugated steel with a guaranteed yield point (.1% offset) not less than 80,000 psi and over 100,000 psi ultimate. Cold-drawn, steel T-wires (transverse wire) not over 6″ c.c. are welded to the deep-corrugated steel in manufacture, and provide temperature reinforcement in the slab, mechanical anchorage and positive shear transfer from concrete to steel. Cofar sheets are hot-dip galvanized, providing complete permanence for normal interior exposures. Extended fire protection beyond any requirement is generally provided by lightweight, modern plaster ceilings, either suspended or directly applied.

During this critical period of defense emergency the material savings alone, made possible by Cofar construction, give it a top priority for designers' consideration.

Cofar saves steel by permitting full range continuous concrete slab design with the steel weight substantially equal to conventional reinforcement.

Cofar saves cement by eliminating non-structural wasted concrete below the reinforcement.

Cofar saves wood by eliminating form sheathing.

Cofar saves labor, time, dollars by a faster simplified construction.

Cut to fit the building frame, Cofar is ready to place — provides immediate working surface for trades.

For complete technical or general information, advice on application and design, review of COFAR designs, estimates and costs for any COFAR project, write Dept. 7-7.

GRANCO STEEL PRODUCTS COMPANY

(Subsidiary of Granite City Steel Co.) • Granite City, Illinois

Continued from p. 29

He has had wide experience in the design of concrete structures. For eight years he was chief engineer of the reinforcing steel department of Connors Steel Co. In 1938 he entered private structural practice, and in 1944 organized the firm of I. E. Morris and Associates, engineers and consultants.

J. D. Lindsay and H. W. Russell

"Illinois Experimental Continuously Reinforced Concrete Pavement after Four Years," p. 653, is described by J. D. Lindsay and H. W. Russell.

J. D. Lindsay, engineer of materials, in charge of the Bureau of Materials, Illinois Division of Highways, Springfield, Ill., is responsible for the inspection and testing of all materials used in highway construction and maintenance under the jurisdiction of the Illinois Division of Highways. He also directs research projects conducted by the Bureau of Materials and is a member of the Illinois Highway Research Council, which has for its objective the encouragement, development and guidance of research in highway problems in Illinois.

Mr. Lindsay is a graduate of the University of Illinois and began work at the Bureau of Materials in 1924. He has been engaged in field inspection, investigational and research work, development and application of procedures used by Illinois in the design and control of portland cement concrete mixtures and development of testing methods and equipment. He was responsible for outlining the research project on continuous reinforcement covered by this paper, supervised the construction of its experimental features and directed the collection and analysis of data on this project.

A registered professional .engineer in Illinois, he is a member of the Illinois Association of Highway Engineers, Sigma Xi, and ACI Committee 325, Structural Design of Concrete Pavements for Highways and Airports.

H. W. Russell, is assistant engineer of materials, Illinois Division of Highways, Springfield, Ill.

Stanton Walker, D. L. Bloem and W. G. Mullen

"Effects of Temperature Changes on Concrete as Influenced by Aggregates," p. 661, was collaborated on for convention presentation by Stanton Walker, D. L. Bloem and W. G. Mullen.

Stanton Walker, director of engineering, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn., Washington, D. C., is a past president of ACI.

An engineering graduate of the University of Illinois in 1917, he was research engineer for PCA from 1917-1926 and since then has been director of engineering of NSGA and NRMCA.

He is a very active member of ACI Committees 115, 214, 613, 617, 621, and ASTM Committee C 1. In addition to ACI, he is a member of ASCE, ASTM, AIMME, AREA, American Foundrymens Assn., and Washington Society of Engineers.

D. L. Bloem is assistant director of engineering, National Sand and Gravel Assn. and National Ready Mixed Concrete Assn.

A graduate of Iowa State College in 1943, he received his professional degree in civil engineering from Iowa State College in 1950. He has been employed by NRMCA and NSGA for the past seven years, first as research assistant, then as associate research engineer in direct charge of the Associations' research facilities at the University of Maryland and, for the past three years has been assistant director of engineering.

Mr. Bloem is a member of ACI and is active on committees of ASTM and HRB.

W. G. Mullen is a former NRMCA Stephen Stepanian Fellow, University of Maryland, 1949-1951.

During the summer of 1942 he was a chainman with PRR and during the summer of 1949 he inspected the construction of a sewage disposal plant.

From 1943-1946 he was a lieutenant in the U. S. Marine Corps and three years later he received his BS in civil engineering from the Virginia Military Institute. In 1951 he obtained his MS in civil engineering from the University of Maryland.

He is a concrete consultant with Madigan-Hyland, engineers, Long Island City, N. Y., and currently is employed on the construction of a new type "floating" concrete pier.

Mr. Mullen is affiliated with ASCE, ARBA and Tau Beta Pi.

Arthur R. Anderson

"Bond Properties in Welded Wire Fabric Reinforced Concrete," p. 681, was presented at the ACI convention by Arthur R. Anderson, partner, Concrete Engineering Co., Tacoma, Wash.

Dr. Anderson has specialized in static and dynamic strain measurements and stress analysis and was retained as consultant on the famous Philadelphia prestressed concrete bridge and also on prestressed concrete girder research work sponsored by the Austin Co., Cleveland, Ohio.

Receiving a BS (cum laude) from the University of Washington in 1934, an MS in 1935 and DSc in 1938 from the Massachusetts Institute of Technology, Dr. Anderson became assistant civil engineer, MIT, 1936-1938. He was a research engineer with Bauer & Schaurte, Neuss Rhein, Germany, in 1938, and design engineer with August Klonne Steel Co., Dortmund, Germany, in 1939. From 1939-1941 he was research associate mechanical engineer at MIT.

Dr. Anderson is a registered engineer in Connecticut and is affiliated with ACI, ASCE, American Welding Society, Society of Naval Architects and Marine Engineers, Society of Experimental Stress Analysis, N. E. Coast Institute of Engineers and Shipbuilders (British), Sigma Xi and Tau Beta Pi.

Ammonium sulfate plant opens in India

A major step toward establishing a heavy chemicals industry in India was marked last month with the opening of a $50,000,000 ammonium sulfate plant in Sindri, Bihar.

In conjunction with the plant are railroad sidings and a marshalling yard consisting of twelve miles of track, and a huge air-conditioned silo with a storage capacity of 90,000 tons of ammonium sulfate. The silo, one of the largest of its kind in the world, is a parabolic arch reinforced concrete structure 150 ft wide, 90 ft high, and 1/8 of a mile long.

Associated with the new plant is a complete township constructed by the Indian Government and a nearby engineering college established by the State Government of Bihar. The college, which operates in close association with the plant, offers courses in chemical, mechanical and electrical engineering.

GET THESE ADVANTAGES BY ADDING

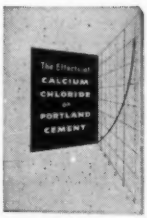
SOLVAY CALCIUM CHLORIDE TO YOUR CONCRETE

- Cuts setting time in half.
- Reduces curing period; permits quicker use.
- Increases both early and ultimate strength.
- Offsets effects of low temperatures.
- Can be used with regular, air entrained, high early, colored or low heat cement.

SOLVAY Calcium Chloride

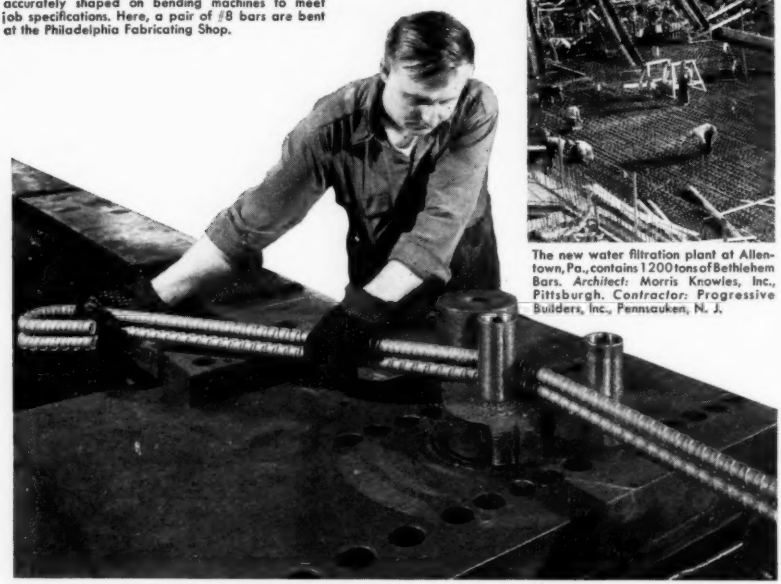
√ ADDS EXTRA STRENGTH
√ SPEEDS OPERATIONS
√ CUTS DELAYS

FREE BOOK GIVES FULL DETAILS. "The Effects of Calcium Chloride on Portland Cement" is offered by Solvay to architects, engineers, contractors — anyone who works with concrete. It is filled with authentic information and answers all your questions about the use of Calcium Chloride in concrete. For your free copy, write on your business letterhead.



SOLVAY SALES DIVISION
Allied Chemical & Dye Corporation
40 RECTOR STREET, NEW YORK 6, N. Y.
Dept. JC-4

Sheared to the proper length, bars of all sizes are accurately shaped on bending machines to meet job specifications. Here, a pair of #8 bars are bent at the Philadelphia Fabricating Shop.



The new water filtration plant at Allentown, Pa., contains 1200 tons of Bethlehem Bars. *Architect:* Morris Knowles, Inc., Pittsburgh. *Contractor:* Progressive Builders, Inc., Pennsauken, N. J.

Bethlehem Bars . . . *fabricated for YOUR JOB*

On most concrete construction jobs, the fabrication of the reinforcing bars you specify should be as important a consideration as the grade of steel or the height and spacing of the bar's lugs.

Bethlehem Reinforcing Bars score on all counts. Only new-billet steel goes into these bars. Their lugs are high and closely spaced to provide firm anchorage in concrete, to help prevent cracking. And Bethlehem Bars are carefully fabricated—cut, bent and tagged—in our own shops to meet exact specifications.

Bethlehem operates shops equipped to fabricate reinforcing bars at Boston, Elizabeth, N. J., Philadelphia, Johnstown, Pa., Sparrows Point, Md., Lackawanna, N. Y., Clearing, Ill., and St. Paul, Minn. For more information about our fabricating service, call the nearest Bethlehem sales office, or write us at Bethlehem, Pa.

Fifteen tons of Bethlehem Reinforcing Bars—bundled, tagged, and with bar lists included—leave the Philadelphia Fabricating Shop for the job-site.

BETHLEHEM STEEL COMPANY
BETHLEHEM, PA.

On the Pacific Coast Bethlehem products are sold by Bethlehem Pacific Coast Steel Corporation. Export Distributor: Bethlehem Steel Export Corporation

★ BETHLEHEM STEEL ★

Bethlehem Reinforcing Bars

Positions and Projects — ACI Members

Committee 115—Research

The assignment of Committee 115—Research, "is to review and correlate research in concrete and reinforced concrete and to consider research methods and objectives." The committee has attempted to fulfill this assignment through the research sessions at the annual conventions, through the distribution of the annual compilation of research projects under way at the various laboratories, and through limited correspondence among committee members.

The annual compilation is divided into three parts. The first part gives the names of all educational institutions that are carrying out concrete research and lists the projects that they are working on. The second part provides similar information for non-educational organizations. In the third part the projects are classified under subject headings. This year, 62 organizations from all parts of the United States and from several foreign countries reported to our committee. These organizations included universities, highway departments, federal departments, cement companies, industrial laboratories, commercial laboratories, and various associations. The reporting organizations listed 396 different projects under way, which cover all phases of research in the fields of aggregates, admixtures, cements, and plain and reinforced concrete. If you did not secure a copy of this compilation at the last ACI convention, you may do so by writing the ACI office.

From the continuing interest of the ACI membership in the activities of our committee, it is evident that real needs are being satisfied. It would be presumptuous, however, to assume that the committee has reached the zenith of its possible usefulness; there must be various ways in which increased service can be rendered. Your reactions and suggestions in this connection are earnestly requested. Since the committee is anxious to have complete coverage of all research projects in its next compilation, all persons having suggestions or knowing of omissions are urged to initiate correspondence at once. No information will be given any publicity without the permission of the correspondent. Address your suggestions and inquiries to:

Professor George W. Washa
Secretary, ACI Committee 115
Engineering Building
University of Wisconsin
Madison 6, Wisconsin

International Organization for Standardization to hold meeting

The American Standards Assn., U. S. member of ISO, announced that the International Organization for Standardization will hold its triennial meeting at Columbia University, New York, N. Y., June 9-21, 1952.

Delegates from the 32 member nations will take part in the ISO General Assembly, which last met in Paris in 1949. Meetings of the ISO Council and a number of its technical committees also will be held during the two-week session.

The ISO is the international clearinghouse for the national standards bodies of most industrial countries. It is recognized by the United Nations as the authoritative channel through which standardization on an international level is carried out.

Levison appointed vice-president and general sales manager

Arthur A. Levison has been appointed vice-president and general sales manager, Blaw-Knox Div., Blaw-Knox Co., Pittsburgh, Pa. Formerly, he was division vice-president in charge of the construction equipment department. Mr. Levison has been an ACI member since 1925.

Symposium on the chemistry of cements

An international symposium on the chemistry of cements will be held at the Royal Institution of Great Britain in London, September 15-20, 1952, under the sponsorship of the Department of Scientific and Industrial Research and the Cement and Concrete Association of England. All interested persons are invited to attend. Information can be obtained through the Secretary of the Symposium, 52 Grosvenor Gardens, London S.W.I., England, or R. H. Bogue, National Bureau of Standards, Washington 25, D. C.

Symposium on the reactivity of solids

An International Symposium on the Reactivity of Solids will be held in Gothenburg, Sweden, June 9-13, 1952, under the sponsorship of the Royal Swedish Academy of Engineering Sciences and the Chalmers University of Technology. All interested persons are invited to attend. Information can be obtained through the Symposium Information Bureau, Chalmers University of Technology, Gibraltargatan 5 J, Gothenburg, Sweden.

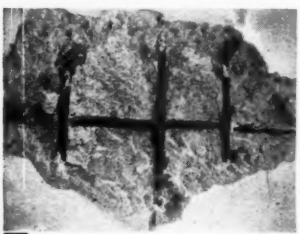
Franks appointed executive vice-president

Carl D. Franks, vice-president for promotion, PCA, was appointed to the newly created position of executive vice-president.

As executive vice-president, Mr. Franks' responsibilities will include exercising the duties of the president of the Association in his absence. He will also have such other duties as may be assigned to him by the president or the Board of Directors. The position of PCA president was left vacant by the death of Frank T. Sheets last November, and has not yet been filled.

"Turner City"

The Turner Construction Co. recently distributed its annual "Turner City," a composite presentation of buildings constructed by the company during 1951. The Turner Construction Co. carried on activities in 40 cities in 14 states. The buildings shown in the "City" involve the construction of 6,828,558 sq ft of floor space, or approximately 156 acres. Henry C. Turner, Sr., founder of the company is one of three living charter members of ACI.



Typical example of spalling. Note corrosion of reinforcing rods exposed by disintegrating concrete.

Reinforced with meshing, the area is restored with gun-applied RESTO-CRETE* by Western Waterproofing Co.

FOR *Lasting* CONCRETE RESTORATION

Specify GUN-APPLIED

RESTO-CRETE*

by WESTERN WATERPROOFING CO.

Sound engineering methods, finest materials, trained technicians and over 35 years' experience assure the job will be done *right* when you specify Western. All work done under contract, fully insured, performance guaranteed. *T. M. Reg.

- Protection from Water Damage (above or below ground, interior or exterior)
- Building Restoration • Tuckpointing

NO MATERIALS FOR SALE • NATIONWIDE SERVICE

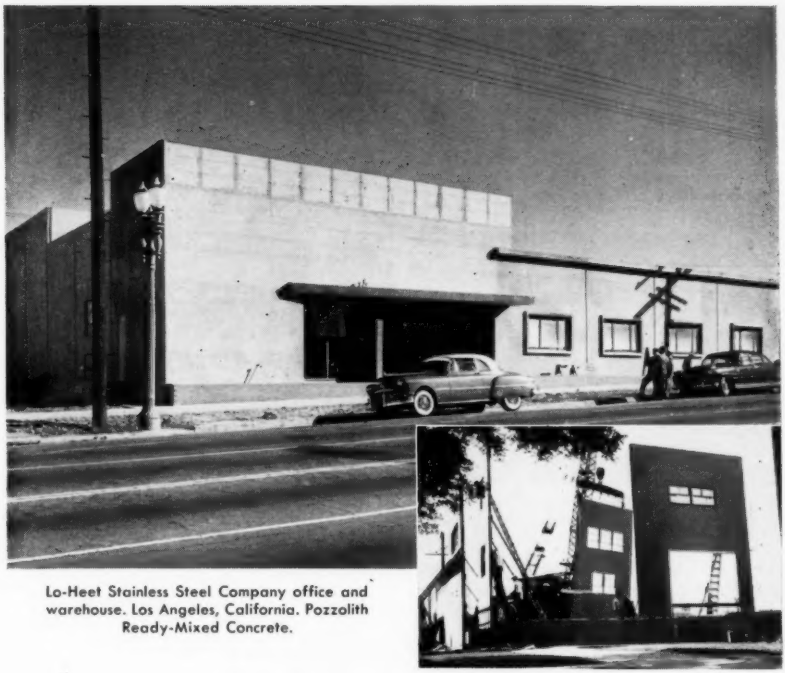
for specific data, write:

WESTERN WATERPROOFING CO.

Engineers and Contractors

1223 Syndicate Trust Bldg. • St. Louis 1, Mo.

A Missouri Corporation Giving Nationwide Service



Lo-Heet Stainless Steel Company office and warehouse. Los Angeles, California. Pozzolith Ready-Mixed Concrete.

Pozzolith Ready-Mixed Concrete is helping builders overcome the problems encountered in tilt-up construction.

Van Wagoner-Swan, Inc., general contractor of the Lo-Heet Stainless Steel Co. office and warehouse, shown above, writes:

"In the construction of our tilt-up concrete units, we have found that Pozzolith definitely decreases shrinkage and the cracking which results from this shrinkage.

"Pozzolith also increases the strength of concrete, allowing us to shorten the time materially between casting and lifting. The decrease in water content with Pozzolith eliminates crazing of surface."

Other recent tilt-up jobs in which construction and cost advantages were obtained with Pozzolith Ready-Mixed Concrete, include: J. W. Greer Co. office and manufacturing plant, Wilmington, Mass.; Northrop Aircraft, Inc. plant, Anaheim, Calif.

Produced by more than 600 leading ready-mix plants, Pozzolith Ready-Mixed Concrete is widely available.

the MASTER BUILDERS Co
CLEVELAND 3, OHIO Subsidiary of American-Marietta Company TORONTO, ONTARIO

Fig. 4 (corrected)—Lateral forces acting on three rectangular columns at right angles to one another

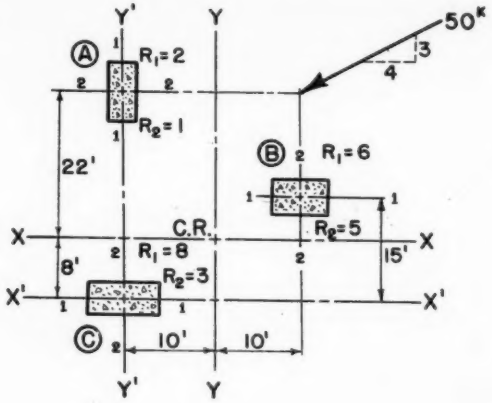
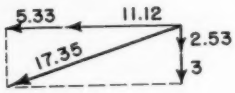


Fig. 4a (corrected) (below)—Resultant force acting on column A in Fig. 4



Errata

The following correction should be made in the paper "Correlation of Shrinkage and Curing in Concrete Masonry Units," ACI JOURNAL, January 1952:

Column 2, Table 2, p. 398, for specimens 10T, 11T and 12T under Test Group B should read "High-*temperature* steam".

The following corrections should be made in the paper "Lateral Force Distribution in a Concrete Building Story," by T. Y. Lin, ACI JOURNAL, Dec. 1951:

The accompanying corrected Fig. 4, 4a, and 7 should be substituted for the corresponding figures in the paper.

The following changes in Example 2, p. 287, should be made:

Step 1—Rigidities having been given, they can be summed up as follows:

$$\Sigma R_y = 2 + 5 + 3 = 10$$
$$\Sigma R_x = 1 + 6 + 8 = 15$$

Step 6—$P_x = 40$ kips $\quad P_y = 30$ kips

$F_1' = (40/15)2 = 5.33$ kips ←

$F_2' = (30/10)1 = 3$ kips ↓

Step 7—Combine the forces vectorially (Fig. 4a). $F = \sqrt{16.45^2 + 5.53^2} = 17.35$. We are likely only interested in the components along the principal axes which are 16.45 and 5.53 as shown.

Last sentence, p. 296, should read:

The magnitude of F can be obtained by substituting in the above formula for F, obtaining $F = 7.26$ kips, the maximum for column A.

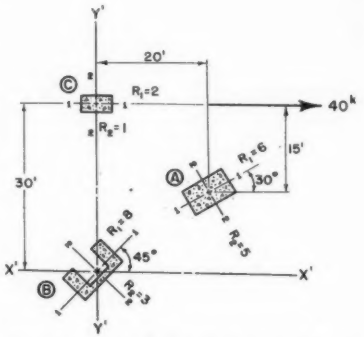


Fig. 7 (corrected)—Principal axes of skewed columns and rigidities along principal axes

ASA Standards to be revised

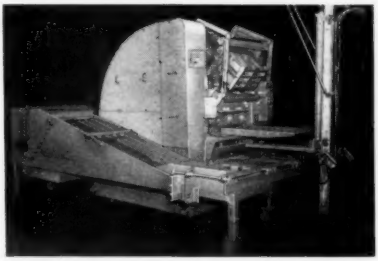
A second draft of a proposed revision of the 1944 edition of "Building Code Requirements and Good Practice Recommendations for Masonry, A41" has been circulated to the sectional committee. Action of the committee is expected to be completed within the next three months.

A proposed revision of the 1945 edition of "Building Loads in Buildings, A58" has been circulated for review and comment by the sectional committee.

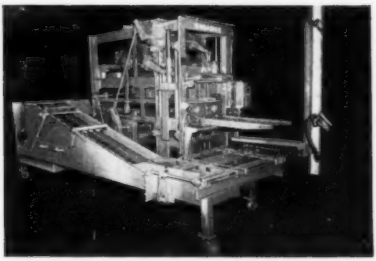
Concrete Profits from Stearns

HIGH PRODUCTION, LOW COST, VIBRATION BLOCK MACHINES FOR CONCRETE PRODUCTS PLANTS OF ALL SIZES

Mixers — Skip Loaders — Turntables — Sinterlite — Genuine Parts — Consulting Engineering Service

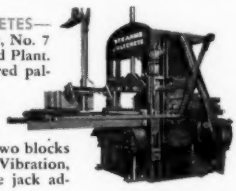


STEARNS 15—For the Larger Plant. Fully automatic. One-man operation. Plain pallets. Power pallet handling. 12-18 eight-inch units per minute, three blocks per cycle. High production and uniform density of units, top to bottom, made possible by Stearns Dual Shaft Vibration under Pressure.

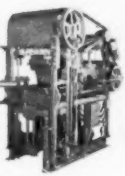


STEARNS 50—For the Medium-Sized Plant. Fully automatic. Plain pallets. One-man operation with power pallet handling. 8-12 eight-inch units per minute, two blocks per cycle. Dual Shaft Vibration with all it means in speed and quality of block.

STEARNS JOLTCRETES—No. 9 for the Larger, No. 7 for the Medium-Sized Plant. Fully automatic. Cored pallets. No. 9—9-11 eight-inch units per minute, three blocks per cycle. No. 7—6 units per minute, two blocks per cycle. Dual Shaft Vibration, eliminating old type jack adjustments.



STEARNS ZIPPERS—For the Smaller Plant. Semi-automatic, power-operated, push-button-controlled. (Manually-controlled, optional.) One-man operation. Cored pallets. 3-5 eight-inch units per minute, one block per cycle. Dual Shaft Vibration. Easily installed, Pressed-top block of Stearns 15, 50 or Joltcrete quality *at a record low cost.*

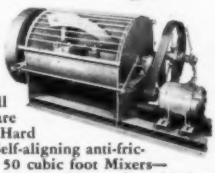


STEARNS SKIP LOADERS—12 and 18 cubic foot Skips operated by 2 h.p., 28-foot by a 3 h.p., and 42 and 50-foot by a 5 h.p. gearhead motor. Push-button control, Convenient charging heights.

STEARNS TURNTABLES—Lower Manpower requirements.

STEARNS SINTERLITE—Write Stearns for further information on machines for converting clay, shale, etc. into lightweight concrete aggregate.

STEARNS MIXERS—12, 18, 28, 42 and 50 cubic feet with long-wearing, replaceable liners. In larger sizes, all members and parts are heavier in proportion. Hard alloy mixing blades. Self-aligning anti-friction bearings. 42 and 50 cubic foot Mixers—air-controlled door optional. Heavy duty friction clutch. Low charging shelf. Gears well guarded. Continuous welded construction. Right or left-hand drive.



STEARNS DUAL SHAFT UNIDIRECTIONAL VIBRATING UNIT IS USED ON ALL STEARNS VIBRATING EQUIPMENT

This simple vibrating system insures uniform and consistent performance and, with lightweight or heavier aggregate, produces block of uniform texture and dimensions, maximum density, high load-bearing strength, clean, sharp edges and corners.

MODERNIZE WITH A STEARNS!

STEARNS

MANUFACTURING COMPANY · INC

ADRIAN • MICHIGAN

30,000 PSI

JUSTIFIED?

IS THERE PRECEDENT? WHAT IS THE RECORD?

For full discussion refer to the article by O. W. Irwin in the January 1952 Journal of the American Concrete Institute, pages 373-378—"CONSERVATION OF STEEL BY DESIGN".

The exchange of advanced ideas always pays off in the free world. American engineers teach many things—but, we can also LEARN from advanced, well established European practice.

UNDER APPROPRIATE CONDITIONS THEIR DESIGN STRESSES ON REINFORCING STEEL ARE:

SWEDEN	**31,500 psi on 57,000 yield point steel**
DENMARK	**30,000 psi on 60,000 yield point steel**
NORWAY	**25,800 psi on 49,780 yield point steel**
GERMANY	**28,400 psi on 50,000 yield point steel**
BRITAIN	**27,000 psi on 54,000 yield point steel**

AND REMEMBER—Under certain conditions the A.C.I. Code permits 28,000 psi on 56,000 yield point wire mesh

RAIL STEEL Concrete Bars have a guaranteed *minimum* yield point of 50,000 psi, it is consistently above 60,000 psi. A working tensile stress of 30,000 psi is SAFE.

U. S. BUILDING CODES SHOULD BE REVISED TO PERMIT THE CONSERVATION OF STEEL AS PRACTICED IN EUROPE.

SAVE STEEL — USE RAIL STEEL EFFICIENTLY

For a complete story of conservation, write for the 35 minute, 16 m.m. kodachrome sound film *"Rail Steel in the World of Today"* Lowell Thomas, *Narrator*

RAIL STEEL BAR ASSOCIATION
38 South Dearborn Street
Chicago 3, Illinois

New Members

The Board of Direction approved 87 Individual, 1 Corporation, 1 Contributing, 17 Junior and 18 Student Memberships for February. With adjustments for losses—resignations, deaths, nonpayments—the total membership reached 5644 on March 1, 1952.

Individual

AMY, HERIBERTO CASTILLO, Guayama, P. R. (Inspector Puerto Rico Housing Authority)
ASCHE, ARTHUR, Chicago, Ill. (Designer, Koppers, Inc.)
BALKE, HARRY A., Cincinnati, Ohio (Cons. Engr.)
BEETH, GUNNAR, Caracas, Venezuela (Designer, Inspector, Instituto Nacional de Obras)
BRUNKEN, CARROL O., Lincoln, Nebr. (Instructor, Univ. of Nebr.)
BUCK, BEN ADAMS, W. Chatham, Mass. (Arch.)
CESPEDES, VICTORINO J., Quezon City, Philippines (Inspector, Manila Dept. of Engrg. & Public Works)
CHANG, DARWIN, Keansburg, N. J. (Designer, Byrne Assocs., Inc.)
CORNELL, RUSSELL WILLIAM, Toronto, Canada (Vice-Pres. Chg. of Canadian operation, Western Waterproofing Co.)
CREPPS, RAY B., Newark, Ohio (Director of Testing Div., Owens-Corning Fiberglas Corp.)
DANFORTH, HERMAN, Honolulu, Hawaii (Designer, Supervisor, Harland Bartholomew & Assocs.)
DANNENBAUM, J. B., Houston, Texas (Cons. Engr.)
DAVIDSON, AARON W., Cincinnati, Ohio (Struct. Designer, Samuel Hannaford & Sons)
DIAZ, JOSUE DIAZ, Guayama, P. R. (Supervisor, Inspector, Nieves Lumber & Constr. Corp.)
DIAZ DE VILLEGAS, RENE, Havana, Cuba
DONLEY, DONALD M., Cleveland, Ohio (Estimator, Sales, Ernest F. Donley's Sons, Inc.)
DOWNIE, WILLIAM W., Halifax, Canada (Arch.)
DOWNING, A. J., Alexandria, Va. (Chf. Engr., Arban & Carosi, Inc.)
DUNCAN, WAYNE E., Honolulu, Hawaii (Struct. Engr., Corps of Engrs.)
EFSEN, AXEL, Copenhagen, Denmark (Prof., Laboratoriet for Bygningsteknik)
EISENBERG, EUGENE R., Boston, Mass. (Manager, Eugene R. Eisenberg, Inc.)
FLEISCHER, HERBERT, Baldwin, N. Y. (Struct. Engr., Roberts & Schaefer Co.)
FUCIK, FRANK M., Chicago, Ill. (Vice-Pres., Genl. Mgr., Waterseals, Inc.)
GOOCH, GEORGE W., Denver, Colo. (Dist. Mgr., Economy Forms, Denver Office)
GORDON, BENNETT T., Philadelphia, Pa. (Cons. Engr., Richardson & Gordon)
GREENBERG, SEYMOUR W., Philadelphia, Pa. (Engr., Estimator, Keystone Concrete Co.)
GUIBERT, GEORGE C., Los Angeles, Calif. (Secy., Genl. Mgr., Super Concrete Emulsions, Ltd.)
HALPIN, JAMES E., JR., Manhasset, N. Y. (Controller, Transit Mix Concrete Corp.)
HANFORD, PHILIP C., JR., Annville, Pa. (Asst. to Dist. Matls. Engr., Pa. Dept. of Highways)
HENRY, JOHN B., Dayton, Ohio (Chf. Engr. of Flexicore Div., Price Bros. Co.)
HERBICH, J. B., (C. E. Aluminum Co. of Canada Ltd.)
HIGUCHI, YOSHIRO, Urbana, Ill. (Student, Univ. of Ill.)
HOBBS, JOHN O., Baltimore, Md. (Field Engr., Portland Cement Assn.)
HOGAN, F. E., Ruston, La. (Assoc. Prof., La. Polytechnic Institute)
HOTTEL, PHILIP F., Wausau, Wis. (Plant Mgr., Purchasing Agent, Wausau Concrete Co.)
HUDSON, FLYNN E., JR., Atlanta, Ga. (Partner, Hudson & Jenkins, Archs. & Engrs.)
HUFFMAN, L. C., Tulsa, Okla. (Plant Supt., McMichael Concrete Co.)
JEFFERIES, EDWARD H., Farson, Wyo. (Chf. Inspector, U. S. Bureau of Reclamation)
JONES, JACK M., Denver, Colo. (Field Super., Economy Forms Corp.)
KANGAS, PELL, Baltimore, Md. (Project Leader, Analyzer, B & O R. R. Co.)
KAWAGUCHI, TERUO, Tokyo, Japan (Concrete Engr., Section Chf., The Japanese National Railways)
KEITH, JAMES N., Riverside, Calif. (Design Engr., Calif. Electric Power Co.)
KENNEDY, J. S., Vancouver, Canada (Mgr., Aerocrete Constr. Co. (B.C.), Ltd.)
KOPYLOV, ALEXEI, Brooklyn, N. Y. (Struct. Designer, Byrne Assocs.)
LEHNEN, JAMES G., Canyon Ferry, Mont. (Matls. Engr., U. S. Bureau of Reclamation)
LEONHARDT, FRITZ, Stuttgart, Germany
LONGLEY, CHARLES S., Atlanta, Ga. (Estimator, Designer, The Southern G. F. Co.)
LUCCHESI, DANIEL ROBERT, Baltimore, Md. (Struct. Designer, Super., Finney, Wolcott & Assoc.)
LYNCH, RICHARD J., E. Orange, N. J. (Plan Examiner, City of E. Orange)
MAYBERG, CHARLES P., Seattle, Wash. (Struct. Designer, Peter H. Hastmark)
MCDOUGALL, CYRILL JOHN, Carnegie North, Australia (Managing Dir., McDougall & Ireland Pty. Ltd.)
MCMURTRY, ALFRED WARNER, Quincy, Ill. (Plant Engr., Moorman Mfg. Co.)
MONTGOMERY, T. J., Cincinnati, Ohio (City Engr., City of Cincinnati)
NAGTEGAAL, GERRIT PAUL, New York, N. Y. (Designer, Hardesty & Hanover)
OKUBO, SHIGEO, Honolulu, T. H. (Asst. Prof., Univ. of Hawaii)
OKUYAMA, SAIZIRO, Tokyo, Japan (Designer, Super., Onoda Cement Co., Ltd.)
PAMEL, JAMES D., Detroit, Mich. (Designer, Cuddie & O'Neill)
PELIKEN, JOSEPH RICHARD, Princeton, N. J. (Genl. Supt., Lewis C. Bowers & Sons, Inc.)
PERIN, JAMES C., Cincinnati, Ohio (Project Engr., Frank Messer & Sons, Inc.)
REINDOLLAR, R. M., JR., Philadelphia, Pa. (Dist. Mgr., Portland Cement Assn.)
RENNER, WALTER ADOLF, Chicago, Ill. (Super. Engr., Roberts & Schaefer Co.)
RIVELAND, A. R., Lincoln, Nebr. (Teacher, Univ. of Nebr.)
RONGVED, PAUL, Bronx, N. Y. (Struct. Engr., Roberts & Schaefer Co.)
ROWE, ROBERT SEAMAN, Princeton, N. J. (Assoc. Prof., Asst. to Dean, Research Assoc., New York Univ.)
SAILOR, SAMUEL, Ithaca, N. Y. (Student, Cornell Univ.)
SANE, VAN RENSSELAER P., Baltimore, Md. (Cons. Engr.)
SCRIVNER, CHARLES A., Huntington, W. Va. (Asst. Mgr. of Sales, Reinforcing Bar Div., W. Va. Steel & Mfg. Co.)
SEDLECKY, ARTHUR F., Grand Rapids, Mich. (Sales Promotion, Designer, F. A. Sedlecky & Co.)
SLADKUS, JOHN H. K., Ottawa, Canada (Engr., Foundation Co. of Canada, Ltd.)
SLATE, FLOYD O., Ithaca, N. Y. (Assoc. Prof., Cornell Univ.)
STROHM, W. E., Santa Fe, N. M. (Super., N. M. State Highway Dept.)
SWANSON, HARRY T., Wayne Pa. (Tech. Service Engr., Univ. Atlas Cement Co.)
SWIHART, GERALD R., Lincoln, Nebr. (Asst. Prof., Univ. of Nebr.)
TAJIRIAN, KHENTIR, Detroit, Mich. (Designer, Drafting, Giffels & Vallet, Inc.)
THAKER, H. J., Nagpur, India (Managing Partner, The National Constr. Co.)
THOMPSON, JOE T., Mounds, Ill. (Designer, Corps of Engrs.)
TREMPER, BAILEY, Sacramento, Calif. (Supervisor, Research Engr., Calif. Div. of Highways)
VALENTI, MICHAEL, Trenton, N. J. (Struct. Engr., Industrial Engrg. Works)
VASLEY, GEORGE T., Frederick, Md. (Struct. Engr., H. K. Ferguson Co.)
VENKATESAN, M. N., Orissa, India (Supervisor, Central Water & Power Comm.)

VIVAS, HECTOR, Caracas, Venezuela (C. E.)
WAEGEMANN, AUGUST ERNEST, San Francisco, Calif. (C. E.)
WASSERMAN, JAIME, Curitiba, Brazil (Civil Constructions)
WHITE, ROBERT T., E. Orange, N. J. (Bldg. Inspector, City of E. Orange)
WHITNEY, BYRON MILLER, Salt Lake City, Utah (Designing, Engr., F. C. Torkelson Co.)
YAMAMOTO, JAMES G., Denver, Colo. (Student)
ZAMORA, L. R. VASQUEZ, Caracas, Venezuela (C. E.)

Contributing

Missouri Portland Cement Co., St. Louis, Mo. (Howard A. Coleman, Dist. Sales Mgr.)

Corporation

Universidad Nacional de Colombia, Bogota, Colombia

Junior

BATCHELDER, GERALD M., W. Lafayette, Ind. (Lab. Testing, Purdue Univ.)
BIDERMAN, SAM, JR., Dallas, Texas (Designer, Drafting, F. W. Chappell & A. M. Brenneke, Cons. Engrs.)
BORGE, ORVILLE E., Portland, Ore. (Lab. Testing, Corps of Engrs.)
COCCHIARELLA, UMBERTO R., Casablanca, French Morocco (Designer, Porter-Urquhart-Skidmore-Owings & Merrill)
DILKS, JOHN ROXBY, Northumberland, England (Designer, Draftsman, British Reinforced Concrete Engrg. Co., Ltd.)
GARANIS, STELIOS, Nicosia, Cyprus (Constr. Supt., Designer, C. Avraamides & C. Michaelides, Contractors)
HASNY, ABD-EL-HADY HUSSIEN, Giza, Egypt (Demonstrator, Ibrahim Pacla Univ.)
CHIRIBOGA, OSCAR L., Izurieta, Quito, Ecuador (Engr., INCA Co.)
KLOTZ, LOUIS H., New York, N. Y. (Struct. Designer, Draftsman, Knappen-Tippetts-Abbett-McCarthy, Cons. Engrs.)
OBADIA B., ISAAC, Caracas, Venezuela (Branch Chf., Vegas & Rodriguez Amengual)
REVESZ, STEPHEN, Birmingham, England (Resident Engr., E. Goldfinger, Arch.)
SAWYER, DONALD ALBERT, Bradenton, Fla. (Field Engr., Parsons, Brinckerhoff, Hall & MacDonald)
STATON, EUGENE L., Chicago, Ill. (Struct. Designer, Draftsman, Naess & Murphy, Archs. & Engrs.)
SUTHERLAND, WILLIAM GILBERT, Madison Heights, Va. (Designer, Estimator, Montague-Betts)
TERP, DONALD JAMES, Casablanca, North Africa (Railroad Design, Porter-Urquhart-Skidmore-Owings & Merrill)
VAN HOUTEN, LEONARD E., New York, N. Y. (Struct. & Foundations Engr., Frederic R. Harris, Inc.)
WALTER, HERMAN J., New York, N. Y. (Designer, Byrne Assocs.)

Student

AMAYA CONDE, OSWALDO, Caracas, Venezuela (Designer, Inspector, INOS)
AQUADRO, RICHARD C., Cambridge, Mass. (MIT)
ARAGAKI, ALFRED YEIKO, W. Lafayette, Ind. (Purdue Univ.)
BINNING, ROBERT C., Urbana, Ill. (Univ. of Illinois)
DUARTE V., HAROLDO, Guatemala, Guatemala
EASTWOOD, WALTER PEARSON, Colorado Springs, Colo. (Colorado College)
HEARD, JOHN F., Denver, Colo. (Univ. of Denver)
KUBBA, MUWAFAG, M. H., Ann Arbor, Mich. (Univ. of Michigan)
LOOMIS, RAYMOND H., Cambridge, Mass. (MIT)
MAURY, CARLOS H., Ft. Wayne, Ind. (Indiana Technical College)
PENEPENT, RICHARD S., Ft. Wayne, Ind. (Indiana Technical College)
PINEDO B., TULIO, Caracas, Venezuela
ROBERTS, JAMES EDWIN, Arlington, Mass. (MIT)
SILVA, MANUEL A., Houghton, Mich. (Michigan College of Mining & Technology)
TELLO, SERGIO L., New York, N. Y. (Univ. of Ill.)
VERES ANTONIO, PETER, Mexico, Mexico
WHALEN, JOHN HUBERT, Three Rivers, Canada
ZWOYER, EUGENE, Urbana, Ill. (Univ. of Illinois)

Honor Roll

February 1—February 29, 1952

The Honor Roll for the current year begins this month. Arturo Obadia B. leads with four credits. Three members are tied for second place—Joseph A. Brunner, George C. Ernst and Howard Simpson—with three credits each.

Greater benefits are derived by increasing membership. More members mean more ideas to be shared—new ideas for discussion.

Arturo Obadia B. (Venezuela)	4
Joseph A. Brunner (N. Y.)	3
George C. Ernst (Nebr.)	3
Howard Simpson (Mass.)	3
Pedro Arismendi A. (Venezuela)	2
J. R. Cassell (Md.)	2
Hunter W. Hanly (Ohio)	2
I. E. Morris (Ga.)	2
Walter H. Price (Colo.)	2
I. S. Rasmusson (Colo.)	2
Theodore O. Reyhner (Colo.)	2
Rafael Rodriguez (P. R.)	2
Saul Shaw (N. J.)	2
Chester P. Siess (Ill.)	2
Willard H. Stevenson (Ind.)	2
Anis Yarid (French Morocco)	2
Robert Zaborowski (N. J.)	2

Tools, Materials, Services

Under this heading note will be made from time to time of producer literature of presumed technical interest (and available from its source for the asking) to ACI users of tools, equipment, materials, accessories and special services.

Masonry drill bits

Designed for drilling ¾- to 5-in. diameter holes in hard masonry, Thunder-Core carbide tipped drill bits are claimed to cut through materials such as concrete with bluestone or granite aggregate and even solid granite.

The Thunder-Core bit is said to make it possible for one man to drill holes quickly and easily in hard masonry with no binding. A new masonry drill bit guide is available upon request.—New England Carbide Tool Co., Cambridge 39, Mass.

Rubber-coated glove

A safety-cuff style glove, coated on the palm surface only with rough finish, natural latex rubber, is reported to cost only a fraction of one cent per man hour. In addition, it is claimed to be extremely flexible, providing complete hand freedom. The rough rubber coating is said to offer a firm nonslip grip, suitable for handling concrete block.—Edmont Mfg. Co., Coshocton, Ohio

No-slip, no-wear floors

All floors in industrial plants, especially loading platforms, receiving and shipping room floors, corridors and runways are subjected to heavy traffic, shock and abrasion. Maintenance men have been demanding tough, durable floors which are not slippery, show no noticeable wear for long periods and involve practically no upkeep costs. Such floor sare said to be possible through the use of Ferem, the "Blue Temper" component.

Ferem component is combined with portland cement and water and does not contain sand and gravel, stone or silica, all of which it completely replaces. Ferem is claimed to interlock by reason of its structural characteristics and grading and to be so ductile and hard that the floors appear as if they had been fused into place. Descriptive literature is available upon request.—A. C. Horn Co. 10th St. and 44th Ave., Long Island City 1, N. Y.

Form clamp

A modern simplified form clamp in 3 parts, weighing only 2 lbs, requiring only ⅜ in. holes in forms or concrete is claimed to be capable of supporting 4 tons. It is said to be easy to use, transport and store and all parts are easily removed with a special puller and saved for re-use.—Meptune Mfg. Co., 3250 E. Olympic Blvd., Los Angeles 23, Calif.

Quick setting cement

Por-Rock, a pourable cement, is claimed by the manufacturer to develop a compressive strength of 4500 psi in one hour. Suggested uses are for setting anchor bolts in concrete floors to secure light and heavy machinery and for spot repairs in concrete floors indoors.—Hallemite Manufacturing Co., 2446 W. 25th St., Cleveland 13, Ohio.

Steel scaffolding data

A revised, 1952 edition of the Waco file folder, a package file including all standard Waco literature, catalog sheets and test data, has been announced by a maker of steel scaffolding equipment. Folders available from Dept. KP, Wilson-Albrecht Co., Inc., 3565 Wooddale Ave., Minneapolis 16, Minn.

Concrete pipe coating

Specifically developed to protect concrete pipe against the chemical attack of sewage and industrial wastes, Rexon Coating No. 2 is claimed to be virtually inert to all acids, alkalies, oils and solvents.

Full details on the characteristics, working properties and methods of application of this air-drying, self-vulcanizing coating are given in Bulletin R-2 which may be obtained upon request.—Hamilton Kent Mfg. Co., 225 Gougler Ave., Kent, Ohio

Vibratory shake-out

This vibratory shake-out is said to provide an effective and inexpensive method of unloading railroad hopper cars, and it is extremely portable and easy to handle. It is a standard type ER TOPDOG external vibrator with a clamping device. The completely enclosed, self-contained vibrator unit delivers 3600 impulses per minute. The clamping device consists of a hydraulic push-pull jack, a bracket to hold the vibrator, and a pair of jaws which are opened and closed by the action of the hydraulic jack.—Vibro-Plus Products, Inc., 54-11 Queens Blvd., Woodside, Long Island, N. Y.

Pneumatic vibrato

Said to be the largest end-mounted pneumatic vibrator made, a new Cannon EM-5-QH vibrator weighs 180 lb. Its 55-lb piston has a stroke of 1⅝ in. and operates at a maximum rate of 650 strokes per minute at an operating pressure of 80 psi. Air-cushioned operation eliminates metal-to-metal pounding for semi-noiseless operation. It is claimed that this feature keeps maintenance at a minimum, that all vibrator parts will last indefinitely since stress causing fatigue failure has been eliminated. The manufacturer stated that by using an extra long stroke, this vibrator sets up a reaction in the entire vibrator assembly thus creating an effective, long amplitude vibration. He further stated that the new 5-in. vibrator was designed for use on large storage bins with a capacity of 75 tons or more.

Full information available on request—Cannon Vibrator Co., 1111 Power Ave., Cleveland 14, Ohio

Membership certificates

Repeated but infrequent requests for membership certificates indicate that enough members may want them to warrant having them made. At its fall meeting, the Board of Direction authorized the Secretary to determine through this notice whether or not the Institute would be justified in going to the expense of having the necessary work done.

It is estimated that certificates suitable for framing can be furnished ACI Members at a cost of not over $3.50. Those interested are requested to clip the attached coupon and mail it to Institute headquarters. *Please do not send money until billed.* If there are not enough interested, the Institute would not be justified in supplying certificates to a few at a loss.

American Concrete Institute
18263 W. McNichols Road
Detroit 19, Mich.

Gentlemen: Please place an order for one membership certificate to be furnished provided enough members buy them to make it worthwhile at a cost not to exceed $3.50.

Signed.....

Portable electric generator

The development of a new portable gas-engine driven electric generator has been announced by the Wincharger Corporation. This Model 1800 generator, in three general ratings—1000, 1250 and 1350 watts at 115 volts, 60 cycles, A.C., is suitable for many power requirements of building contractors, portable power tools, etc. This generator is said to be rated conservatively and is ruggedly built, being equipped with sealed ball bearings.

It is claimed that the belted construction cushions shock to the generator, reduces vibration and increases the generator life. The generator being mounted alongside the engine locates it out of the path of the hot air from the engine cooling system, thus promoting cooler generator operation.

The unit is equipped with a universal mounting base which permits mounting on any one of several popular makes of engines. The generating unit may be purchased completely equipped with engine or may be purchased without an engine. The generator alone without the engine mounting base is also available.

Free literature is available from the manufacturer upon request.—Wincharger Corp., 2110 E. 7th St., Sioux City 2, Iowa

New specifications

New standard specifications for vermiculite plastering and acoustical plastic have been issued by the Vermiculite Institute. The plastering specifications are in general agreement with American Standard Specifications for Gypsum Plastering, ASA No. A42.1-1950.

The 12-page booklet covers the application of vermiculite plaster base coats to all types of lath and masonry bases, solid partitions, and standard finish coats, including the new vermiculite trowel finish.

Of general interest to architects and contractors is the claim that vermiculite acoustical plastic can be applied directly to monolithic concrete ceilings thus requiring no base coat.

Detailed suggestions for best results in plastering over masonry, solid partitions, and metal lath ceilings are given in the booklet.—Vermiculite Institute, 208 S. LaSalle St., Chicago 4, Ill.

Adjustable steel shores

A 4-page booklet, giving detailed information and showing typical applications of patented adjustable steel shores, points out construction economies resulting from the use of adjustable steel shores due to the elimination of cutting, wedging and nailing that must be done where conventional wood shoring methods are employed. Due to their adjustable feature, they are said to be readily and easily adjusted to any height desired and therefore can be used over and over again on any job or different shoring application.

The booklet highlights some of the outstanding features of the steel shore—one of the most important being a unique self-cleaning feature by which dirt, encrusted concrete and grit are cleaned off the threaded surface with the turn of a handle. Copies of the booklet are available upon request.—Acrow, Inc., 510 N. Dearborn St., Chicago 10, Ill.

200-ton compactor

A new 200-ton rubber tired roller has been designed and built for use in testing fills on air base runways and for other fills where extreme compaction is necessary.

The roller is designed in two halves, each half being provided with two tires and so arranged that each half will oscillate independently, thus following the undulation of the fill exactly. The rear of the roller is equipped with an extra tongue, so that propelling tractors may be used both front and rear if necessary. At the rear of the roller is also furnished a 15 HP, 4 cylinder, gasoline engine-driven hydraulic pump which supplies 5000 psi for a 10 in. hydraulic cylinder. —Shovel Supply Co., Dallas, Texas

Masonry drill guide

Four-page booklet describes types of drills and recommended drilling procedure for precision drilling in hard concrete, soft and medium hard masonry materials and in hard and very hard masonry.—New England Carbide Tool Co., Inc., 60 Brookline St., Cambridge 39, Mass.

Metal thresholds

Design and application of thresholds are described in a 28-page, pocket size book. Illustrated are various types of abrasive cast, extruded and rolled steel thresholds with cross-section drawings and typical installation sketches.—Wooster Products, Inc., Dept. C, Wooster, Ohio

Proceedings of the First U. S. Conference on Prestressed Concrete

The Massachusetts Institute of Technology has made available the Proceedings of the First U. S. Conference on Prestressed Concrete. This volume of Proceedings is a compilation of the 38 papers presented at the Conference, August 14-16, 1951, dealing with applications, materials, design and research pertaining to prestressed concrete. (See p. 693, April Current Reviews.)

Purdue publishes cement and concrete bibliography

The Engineering Experiment Station of Purdue University has announced the publication of "Comprehensive Bibliography of Cement and Concrete 1925-1947" by Floyd O. Slate.

This 491-page bibliography contains more than 40,000 references which substantially represent the world's scientific literature in the field of cement and concrete. A few of the major subdivisions which illustrate the variety of subjects covered are: Admixtures, aggregates, mortar and concrete coatings, reinforced concrete, ready-mixed concrete, precast concrete and winter concreting.

For further information, see p. 696.

Major Carl L. Sitter, USMC, Medal of Honor

THE HILL WAS STEEP, snow-covered, 600 feet high. Red-held, it cut our lifeline route from Hagan-ri to the sea; it had to be in our hands.

Up its 45-degree face, Major Sitter led his handful of freezing, weary men —a company against a regiment! The hill blazed with enemy fire. Grenade fragments wounded the major's face, chest, and arms. But he continued heading the attack, exposing himself constantly to death, inspiring his men by his personal courage. After 36 furious hours the hill was won, the route to the sea secured. Major Sitter says:

"Fighting the Commies in Korea has taught me one thing—in today's world, *peace is only for the strong!* The men and women of America's armed forces are building that strength right now. But we need your help—and one of the best ways you can help us is by buying Defense Bonds.

"So buy bonds—and more bonds—starting *right now*. If you at home, and we in the service, can make America stronger *together*, we'll have the peace that we're all working for!"

★ ★ ★

Remember, when you're buying bonds for national defense, you're also building a personal reserve of cash savings. Remember, if you don't save *regularly*, you generally don't save at all. Sign up today in the Payroll Savings Plan where you work, or the Bond-A-Month Plan where you bank. For your country's security, and your own, buy Bonds now!

Peace is for the strong...
Buy U. S. Defense Bonds now!

ALPHABETICAL LIST OF ADVERTISERS

(Page numbers refer to News Letter)

Baldwin-Lima-Hamilton Corporation....30
Bethlehem Steel Company....34
Chain Belt Company....13
Dewey and Almy Chemical Company....28
Granco Steel Products Company....31
Intrusion-Prepakt, Inc....2
Laclede Steel Company....20
Lone Star Cement Corporation....Inside front cover
Master Builders Company....37
Rail Steel Bar Association....40
Solvay Sales Division, Allied Chemical and Dye Corporation....33
Stearns Manufacturing Company, Inc....39
Charles R. Watts and Company....15
Western Waterproofing Co. of Missouri....36

The Institute assumes no responsibility for the claims of advertisers. The advertiser is made responsible in the belief that his place in the field will be determined by the public's ultimate measure of his exercise of that responsibility.

DISCUSSION

Discussion closed January 1, 1952

Sept. Jl. '51

Earthquake Resistant Design Considerations—R. R. Martel

Comprehensive Numerical Method for the Analysis of Earthquake Resistant Structures—Charles S. Whitney, Boyd G. Anderson and Mario G. Salvadori

Multistory Buildings Designed to Resist Earthquakes—John J. Gould

Lightweight Concrete for Lower Construction Costs—J. A. Murlin

Designing for Continuity in Prestressed Concrete Structures—Alfred L. Parme and George H. Paris

Pumice—Lightweight Aggregate—Leslie I. Neher

Hydration Products Formed in Cement Pastes at 25 to 175 C—George L. Kalousek and Milton Adams

Discussion closed February 1, 1952

Oct. Jl. '51

Advances in Precast Floor Systems—F. N. Menefee

Manufacture and Use of Machine-Made Precast Structural Elements—A. G. Streblow

Tilt-up Construction in Western United States—F. Thomas Collins

Diagonal Tension in Reinforced Concrete Beams—Arthur P. Clark

Coral and Salt Water as Concrete Materials—John G. Dempsey

Load Carrying Capacity of Dowels at Transverse Pavement Joints—Henri Marcus

Discussion closed March 1, 1952

Nov. Jl. '51

Tilt-Up Construction Costs—F. Thomas Collins and Earl Bennetsen

Effect of Temperature and Surface Area of the Cement on Air Entrainment—E. W. Scripture, Jr., S. W. Benedict and F. J. Litwinowicz

Foundation for a Large Turbogenerator—Paul Rogers

Measurement of the Distribution of Tensile and Bond Stresses Along Reinforcing Bars—R. M. Mains

Insulation for Protection of New Concrete in Winter—L. H. Tuthill, R. E. Glover, C. H. Spencer and W. B. Bierce

Curing of Concrete*

SYNOPSIS

Techniques for curing concrete must consider all the conditions, both natural and artificial, that affect the extent and rate of cement hydration. Well-cured concrete does not just happen; it is the result of careful control of the moisture content and temperature of the concrete.

A survey of pavement curing methods in the 48 states indicates fairly uniform practice in initial curing methods but wide variations in final curing methods and total time of application. The initial curing method most widely permitted is covering with saturated burlap. For final curing, saturated cotton, felt or jute mats, membrane compounds, waterproof paper, saturated earth or ponding and covering with hay or straw are permitted. Length of curing period varies from 72 hours to 7 or more days.

A summary of the factors which relate especially to the curing of concrete in building construction emphasizes where and how these differ from some of the problems encountered in other fields. Current practices, especially with regard to form removal and to temperature and moisture control in cold weather are discussed.

Problems in curing mass concrete are similar to those for smaller sections with the addition of consideration of heat of hydration. Methods used must control the temperature differential between the face and interior of the massive sections. Some remedial measures are creating low temperature in the concrete when placing, limiting the height and rapidity with which the lifts are placed, use of embedded cooling systems or using steel forms which help dissipate the heat. A summary of the recommended curing practices for mass concrete based on the most recent report of Committee 612 is included.

A discussion of membrane curing methods for concrete canal linings points out that white pigmented compound is preferred. Good material, proper time of application, adequate film thickness, and uniform coverage are important.

Standard laboratory and field curing methods are summarized. The influence of economy in the standardization and establishment of acceptable curing specifications is discussed. Use of calcium chloride as a surface treatment or integrally for curing purposes, is explained.

Introduction

By A. G. TIMMS†

CURING DEFINED

Curing, as applied to the making of concrete, covers all the conditions both natural and artificially created that affect the extent and the rate of

*Presented at a round-table discussion at the ACI 48th annual convention, Cincinnati, Ohio, Feb. 26, 1952. Title No. 48-46 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Materials Engineer, Bureau of Public Roads, Washington, D. C.

hydration of the cement. Many concrete structures are cured without protection of any kind by merely allowing them to harden in the outdoor air with the surface exposed to sun, wind, and occasionally rain. Curing under these conditions is unreliable and frequently harmful. Premature surface drying may cause excessive cracking since the concrete has not attained sufficient strength to resist the tensile forces developed as the result of rapid moisture loss. By creating favorable curing conditions through control of moisture and temperature, the rate and extent of the hydration is enhanced with resultant improvement in strength, hardness, and watertightness.

MOISTURE CONTENT

In the art of concrete making, curing refers to the various means employed to control moisture content or temperature of the concrete or both. In practice, artificial curing is the use of methods for conserving moisture within the concrete or furnishing additional moisture to replace that lost by evaporation. In general, little attention is paid to the control of temperature during the curing period except in special cases such as steam curing or cold-weather protection.

The mixing water used in making concrete serves two purposes: first, to facilitate placement of concrete, second, to furnish water to react with the portland cement to produce hydration products. Investigators have shown that the amount of water required for hydration is relatively small compared with the total volume required for mixing. T. C. Powers[1] has discussed some features of the chemistry and physics of cement hydration in relation to curing.

TEMPERATURE

In the early days of concrete making, speed in construction was not as important as it is today and the forms were left in place longer. The heavy wood forms used had greater insulating value than the present thin plywood sheets and metal pans. Also concrete paving slabs were usually protected by a wet earth cover or by ponding. These methods not only provide water but due to the high specific heat of water, also provide insulation against sudden temperature changes. Burlap curing was introduced for the early protection of exposed surfaces of pavement slabs until they had hardened sufficiently to permit ponding or the placing of wet earth cover. The scarcity of burlap during and since World War II and its present high cost have tended to decrease its use for early curing. Cost has also been a factor in discouraging the use of cotton mats, a type of curing that not only prevented evaporation, but provided excellent insulating properties.

WATERTIGHTNESS

F. R. McMillan,[2] in 1929, pointed out the vital influence of curing on watertightness. He showed that in some cases the quantity of combined water in

concrete could be doubled by thorough curing under favorable temperature conditions, thus greatly improving the degree of watertightness. Further, for a given water-cement ratio, curing is the major factor in developing the potential strength of the concrete. The following is McMillan's opinion on the value of moist curing; "Improved curing to force more and more of the water into the permanent solids of the mass has been emphasized. This is important, and the test data quoted, as well as experience, have shown that it offers the best opportunity for better concrete under our present methods."

As has been pointed out, moisture is necessary to the hardening process of portland cement and constitutes the most essential factor in curing. When sufficient moisture is not present the curing process stops and no improvement in strength or other properties will be obtained.

METHODS

Methods adopted from time to time for curing concrete may be classified as (a) those that supply water throughout the early hydration process and tend to maintain a uniform temperature, such as ponding, sprinkling, application of wet burlap or cotton mats, wet earth, sawdust, hay or straw, and (b) those designed to prevent loss of water but having little influence in maintaining a uniform temperature, as waterproof paper or impermeable membranes. Frequently a curing process is employed that combines the two.

The problems of curing involve not only supplying moisture and favorable temperature conditions but also such factors as the optimum time to start application of curing, duration of the time moisture is made available and where durability is of prime importance, the desirability of a drying period following moist curing.

TIME OF APPLICATION

The optimum time to start moist curing is not easy to determine. It appears, in pavement practice, that the concrete should be allowed to subside in the forms and all free moisture disappear from the exposed surface before application of moisture or other forms of curing. This optimum time is even harder to determine for air-entraining concrete where frequently little subsidence occurs and very little free water is observed on the exposed surface. Sometimes the nature of the curing treatment dictates how soon after placing the concrete the curing may be applied. For pavement it is frequently specified that wet burlap be placed as soon as the surface is hard enough not to be marked by the burlap.

Too rapid drying of fresh concrete in the very early stages may result in crazing or cracking of the surface. Under extremely unfavorable conditions severe cracking may occur. The ability of concrete to harden is not destroyed by drying, since rewetting will cause the hardening to be resumed, but, if early drying has resulted in crazing of the surface, the fine hair cracks will not be healed by subsequent curing. M. A. Swayze[3] has stated that by withholding curing water until the concrete is nearing its maximum temperature,

the later thermal shrinkage can be decreased. However, evaporation from the fresh concrete surface must be prevented until the concrete reaches maximum temperature. The Germans, in constructing the Autobahnen, maintained this condition by protecting the fresh concrete pavement surface by movable canvas tents. In this connection T. C. Powers[1] has observed in the laboratory that if curing water is not supplied during the initial stages of rapid hydration some self-desiccation of the paste will result. Under these conditions subsequent applications of curing water will not restore the maximum degree of saturation.

MEMBRANE CURING SPECIFICATIONS

Since the start of World War II, liquid membrane-forming curing materials have been used extensively in the highway field. Their use is based on the theory that if the freshly mixed concrete is covered with an impervious seal, sufficient water will be retained for hydration. Many investigators have struggled with the problem of how to evaluate adequately these liquid membrane-curing materials. No national requirements for measuring the effectiveness of liquid membrane-forming curing compounds have been developed. A water retention requirement is suggested in Appendix 2 of the Highway Research Board Wartime Road Problems Bulletin No. 1, Curing Concrete Pavements. The U. S. Army Engineers also have a water retention requirement in their specifications. A subcommittee of ASTM Committee C-9 is revising the method of making the water retention test and preparing a specification for liquid membrane-forming curing materials applying a water retention test as a measure of effectiveness.

COLD WEATHER CURING

Hydration reactions proceed more rapidly at higher temperatures, and hardening of concrete is greatly retarded as the temperature approaches the freezing point of water. Temperatures below 50 F are considered unfavorable for curing, particularly if high early strength is desired. When concreting at temperatures below 70 F it is desirable to use curing methods which tend to maintain suitable temperatures during the hardening period.

"Recommended Practice for Winter Concreting Methods," ACI JOURNAL, Sept. 1948, gives good general rules covering curing temperature which are quoted as they are applicable to this discussion.

> Newly placed concrete made with normal cement should be kept at a temperature of not less than 50 F for 7 days or not less than 70 F for 3 days. It is good practice to maintain a temperature of at least 40 F for the next 4 days. At the end of the curing period, artificial heating should be discontinued and housing removed in such a manner that the fall in temperature at any point in the concrete will not exceed 40 F in 24 hours.
>
> Concrete made with high early strength cement should be kept at not less than 70 F for 2 days or not less than 50 F for 3 days.
>
> The surface temperature of the hardened concrete should not be permitted to exceed 100 F at any time during the curing period. The methods of protection and curing should be such that the loss of moisture from the concrete is not excessive during the curing period.

Artificially heating the concrete introduces some danger of drying and interferes with the absorption of moisture. Undoubtedly many have noticed in cold weather concreting of structures where salamanders are used to maintain adequate temperature conditions that concrete adjacent to these heating devices frequently shows signs of premature drying such as crazing cracks or a powdery condition of the surface.

There is evidence that prolonged moist curing of air-entraining concrete without an intervening drying period will tend to fill the voids with water and reduce its potential durability. This was demonstrated in tests conducted at the research laboratories of the Bureau of Public Roads.[4] In the case of air-entraining concrete for severe exposure a drying period should be employed following the initial moist curing period to insure maximum durability.

Because of the importance of curing on the properties of concrete, it appears sound practice to require a certain minimum period of controlled moisture and temperature.

REFERENCES

1. Powers, T. C., "A Discussion of Cement Hydration in Relation to the Curing of Concrete," *Proc.*, Highway Research Board, 1947, V. 47, p. 178.

2. McMillan, F. R., *Basic Principles of Concrete Making*, McGraw Hill Book Co., Inc., New York, N. Y., 1929.

3. Swayze, Myron A., "Finishing and Curing: A Key to Durable Concrete Surfaces," ACI JOURNAL, Dec. 1950., *Proc.*, V. 47, p. 317.

4. Timms, A. G., "Freezing and Thawing Tests of Concrete Containing Oregon and Washington Aggregates," *Public Roads*, Oct. 1951.

Current Practices for Curing Concrete Pavements

By D. L. ROBINSON*

PAVEMENT CURING SPECIFICATIONS SURVEY

There are many methods and combinations of methods for curing concrete pavement slabs. Water curing, to replace water lost by evaporation, may be accomplished by periodic sprinkling, by ponding, or by saturated coverings of earth, hay, straw, cotton and jute mats, and multi-layered fabrics. Evaporation may be prevented by covering with impervious paper or impervious coatings or membranes. Calcium chloride may be used to accelerate the gain in strength and reduce the period of curing or may be used as a surface application, to attract moisture from the atmosphere.

For this report, current practices for curing concrete pavement are taken from specifications of state highway departments, where available, and the *Charted Summary of State Highway Specifications* prepared in 1951 by the

*Senior Engineer, Bureau of Materials, Missouri State Highway Dept.

Portland Cement Assn., but not yet published. This PCA publication is considered authoritative since it was developed by a study of highway specifications, and interviews with highway department personnel. Being a summary, it is sometimes difficult to interpret details, particularly in regard to curing procedure required prior to application of the final curing method.

It is to be understood that *current practice* and *methods permitted* by specifications are not necessarily the same, since some methods may be permitted which are seldom used. Since specifications cannot reflect the popularity or extent of use of the various methods, it is necessary to consider for the purpose of this report, that any method permitted is used to some extent.

ANALYSIS OF SURVEY

Initial curing—saturated burlap

From the information collected and tabulated it is found that, excluding Nevada which does not seem to have a specification for portland cement concrete pavement in its standards, the most widely specified method of initial curing is covering with saturated burlap (Table 1). Only two states, Arizona and California, specify another method exclusively which is fog sprinkling. Montana permits burlap or wetting. Preliminary burlap curing is required by some states, not only prior to final curing by earth cover or ponding but also before the application of saturated mats, waterproof paper or sealing membranes.

Final curing—saturated mats

For final or for complete curing, covering with saturated cotton, felt or jute mats or two thicknesses of burlap is the most universally permitted method. It is permitted by 44 states. Six of these require preliminary burlap covering for various periods up to 24 hours, while 29 states permit application of the mats as soon as the concrete has hardened sufficiently to prevent marring the surface. Only three states, New Hampshire, Minnesota and Utah, do not mention this type of curing. Nebraska and South Dakota permit burlap but not mats. Twenty-two states require mats to be made of cotton and naturally the cotton producing states predominate in this classification.

TABLE 1—CONCRETE PAVEMENT CURING METHODS. NUMBER OF STATES REQUIRING INITIAL CURING WITH BURLAP IN CONJUNCTION WITH OTHER PERMITTED METHODS

Methods permitted	Number of states			
	Burlap initially required with other methods listed			Curing method not mentioned
	Yes	No	Unknown	
Earth or ponding	31	3	—	13
Hay or straw	25	2	—	20
Felt mats	2	15	3	27
Cotton mats	6	29	7	5
Burlap	26	—	—	21
Paper	7	25	8	7
Membrane	2	28	4	13

Waterproof paper

Covering with waterproof paper is permitted by 40 states, seven of which specifically require initial burlap curing. They are Pennsylvania, Georgia, Alabama, Kentucky, Iowa, South Dakota and Colorado. Eight more states may require initial curing; information is not sufficiently specific

to classify them otherwise. Seven rather widely distributed states do not mention waterproof paper in their specifications; they are Delaware, South Carolina, Indiana, Michigan, Kansas, Oklahoma and Wyoming.

Saturated earth or ponding

Thirty-four states permit final curing by covering with saturated earth or by ponding. All but Arizona, California, and Montana require burlap covering for a preliminary curing period. Thirteen states do not mention earth or ponding; seven of these are in the New England and Mid-Atlantic regions, others are in the Mid-west.

Membrane sealing compound

Thirty-four states permit membrane sealing compounds, ordinarily of the clear or transparent type. Of these, 11 states permit membrane curing only by special provision or with specific permission. It appears that Tennessee and Iowa require initial burlap curing and 4 more states may require it. Maryland, Vermont, Iowa and Utah permit black or bituminous types. Iowa permits them only if immediately covered with whitewash. California, Arizona and Washington specify pigmented membranes and Michigan and Colorado permit them. Thirteen states do not mention membrane curing methods; four of them in the New England region, the remainder distributed among all regions except the Pacific and southeast Central region.

Hay or straw

Covering with saturated hay or straw seems to be the least widely permitted method in general use for final curing. Such coverings are permitted by 27 states and all but Illinois and Montana require preliminary curing with burlap. Illinois permits sprinkling and Montana permits wetting, probably also by sprinkling. Dry hay or straw covering is widely used for protection during cold weather. The 20 states which do not mention this type of covering as a curing method are located primarily in the south Atlantic, southwest Central and Pacific regions.

Summary

Other miscellaneous methods of curing permitted by a few states are surface application of calcium chloride by Maryland, Rhode Island and Missouri, sodium silicate by West Virginia and Missouri, irrigation by Washington, sawdust by Oregon and continuous or periodic sprinkling by Washington. All methods are summarized in Table 2.

Thus covering with saturated fabrics or mats is the most widely permitted method for curing concrete pavement, followed in order by impermeable paper, earth or ponding, sealing membranes, and hay or straw. There are few marked regional preferences although some are noted and in actual practice there are probably many more. Some rather substantial differences are evident in details of control of curing, particularly in the amount and time of water application and time of application for mats and membrane.

TABLE 2—PAVEMENT CURING METHODS PERMITTED BY VARIOUS STATES AND

REGION	Earth or ponding			Hay or straw			Felt mats			Cotton mats		
	Yes	No	Unknown	Yes	No	Unknown	Yes	No	Unknown	Yes	No	Unknown
NEW ENGLAND					ME				MASS		CONN ME	MASS
	MASS			MASS								
	RI			RI				RI			RI	
				VT							VT	
MIDDLE ATLANTIC				NJ				NJ			NJ	
				NY				NY			NY	
								PA			PA	
SOUTH ATLANTIC	DEL			DEL					DEL			DEL
	FLA										FLA	
	GA									GA		
	MD										MD	
	NC											NC
	SC			SC				SC			SC	
	VA						VA			VA		
	W VA			W VA							W VA	
EAST NORTH CENTRAL					ILL			ILL			ILL	
	MICH			IND				IND			IND	
	OHIO			MICH				MICH			MICH	
	WISC			OHIO				OHIO			OHIO	
								WISC			WISC	
EAST SOUTH CENTRAL	ALA			ALA						ALA		
	KY			KY							KY	
	MISS						MISS			MISS		
	TENN			TENN				TENN			TENN	
WEST NORTH CENTRAL	IOWA			IOWA				IOWA		KAN	IOWA	
	KAN											
	MO			MO				MO			MO	
	NEB			NEB							ND	
	SD			SD								
WEST SOUTH CENTRAL											ARK	
	LA											LA
	OKLA										OKLA	
											TEX	
NORTH MOUNTAIN	MONT			MONT				MONT			MONT	
	IDA			IDA								IDA
	WYO			WYO								WYO
	COLO			COLO							COLO	
	UTAH			UTAH								
SOUTH MOUNTAIN	NM			NM				NM			NM	
	ARIZ										ARIZ	
PACIFIC	CAL										CAL	
	ORE								ORE			ORE
	WASH									WASH		
TOTALS	31	3	0	25	2	0	2	15	3	6	29	7
METHOD NOT PERMITTED	13			20			27			5		

NOTES
(A) Also permits $CaCl_2$ surface application.
(B) Also permits sodium silicate.
(C) May use spray for initial curing period.
(D) Also permits wet sawdust.

LENGTH OF CURING PERIOD

There seems to be little agreement among the states on the proper length of the curing period, except that 72 hours is the minimum required. Of the 34 states permitting earth or ponding, 11 require saturation for only 3 days, 14 require a saturation period of 4 to 6 days and 9 require it for 7 days or longer. Twenty-four of the 42 states permitting the use of saturated mats

WHETHER OR NOT OTHER METHODS ARE REQUIRED FOR THE INITIAL PERIOD

Burlap	Impervious paper			Membrane				Specification available	Notes
	Yes	No	Unknown	Yes	No	Unknown	Types		
CONN		CONN			..				
..		ME			..				
MASS		MASS				MASS			
..		NH			..			1948	
RI		RI			..			1946	(A)
..		VT				VT	Bit?	1948	
..		NJ			NJ		Non Bit		
NY		NY			NY sp		Clear	1947	
..	PA				..			1950	
DEL		..			..				
FLA		FLA			FLA		Clear	1950	
GA	GA				GA sp		Clear	1947	
MD		MD			MD		Clear or Bit	1948	(A)
NC			NC		NC sp				
SC		..			SC sp		Clear		
VA		VA			VA sp			1947	
W VA			W VA			W VA	Clear		(B)
ILL		ILL			..			1949	(C)
IND		..			..			1946	
MICH		..			MICH		Wh or Clear	1950	
..		OHIO			OHIO		Clear	1949	
WISC		WISC			WISC		Clear	1951	
..	ALA				ALA		Clear	1950	
KY	KY				KY sp			1945	
..		MISS			MISS sp		Clear	1950	
TENN			TENN	TENN			Clear		
IOWA	IOWA			IOWA			Bit Wh or Clear	1948	(G)
..		..			..			1951	(F)
..		MINN			MINN			1947	
..		MO			MO		Clear	1950	(A) (B)
NEB		NEB			NEB		Clear		
ND		ND			..			1949	
SD	SD				SD		Clear	1946	
..			ARK		..				
LA			LA		LA sp		Clear		
..		..			..				
..		TEX			TEX sp		Clear	1951	
MONT		MONT			MONT		Clear		(C)
..			IDA		IDA sp		Clear		
WYO		..			..				
..	COLO				COLO		Wh or Clear	1948	
..			UTAH		UTAH sp		Bit		
..		NM				NM	Clear		
..		ARIZ			ARIZ		Wh	1948	
..		CAL			CAL		Gray	1949	
ORE			ORE		ORE		Clear		(D)
..		WASH			WASH		Wh	1948	(E)
26	7	25	8	2	28	4			
21		7			13				

(E) Also permits ten day sprinkling, also irrigation.
(F) Mats permitted for urban projects only.
(G) Bit membrane permitted only if whitewashed.
sp Permitted by special provision or under certain conditions only.

permit removal after 72 hours and 7 require them to remain in place for 7 days or longer. Of the 40 states permitting paper curing, 25 permit removal after 72 hours and 6 require it to remain in place for 7 days or longer (Table 3).

Kansas has made the most drastic change in its curing procedure. It now requires a 4-in. saturated earth cover for 10 days, excluding the initial burlap curing. The earth cover must be left in place for an additional 10 days and

TABLE 3—LENGTHS OF CURING PERIODS REQUIRED. NUMBER OF STATES REQUIRING THE LISTED TIME FOR EACH OF THE PERMITTED METHODS

Methods permitted	Number of states			
	Time			Curing method not mentioned
	72 hours	4 to 6 days	7 or more days	
Earth or ponding	11	14	9	13
Hay or straw	5	14	8	20
Mats	24	11	7	5
Burlap	16	7	3	21
Paper	25	9	6	7

the pavement closed to traffic for a total of 30 days after placing. Cotton mats may be used on city sections under the same conditions except that pavement may be opened after 20 days. No other methods are permitted. Montana requires 14-day curing for hay, straw, mats, burlap and paper. Washington requires 10 days for earth, ponding, mats and paper and Wyoming requires 8 days for earth, ponding, mats and burlap. Colorado requires paper to remain in place for 8 days.

CURING METHODS

Economy

The selection of curing methods is a matter of economics, just as is the selection of mix proportions and quality of aggregates. Thus, states properly permit several methods from which a contractor may select the one that will be the least costly and will interfere least with his other operations, but which, in the opinion of the officials, still will give suitable results under the prevailing climatic conditions.

Curing methods have been investigated by many states and agencies and there are many excellent reports on these investigations, some fairly recent. During 1930 Missouri made a field study of curing using 24 and 72 hours of wet burlap, ponding, black membranes with and without whitening coatings, sodium silicate, waterproof paper and surface application of calcium chloride. No quantitative comparisons of strengths could be made as all were within the range of probable error of tests on the cores. The difference in strengths between the best method used (24-hour wet burlap followed by ponding) and the poorest (24-hour wet burlap with no subsequent artificial curing) did not exceed 15 percent. The indications were that these methods provided satisfactory moisture retention and strengths and it was concluded that for Missouri's climatic conditions all of them except the black membrane type without whitening yielded acceptable results. An undesirable increase in frequency of cracking accompanied the use of uncoated black membranes.

Effectiveness of membrane curing

Since membrane curing was simple to use and less costly than most other methods, the clear and light pigmented types were developed and, when permitted without initial burlap curing, came into almost exclusive use by contractors. It had early been considered that if some 50 percent of the mixing water could be retained, the cement would be adequately hydrated but such intensive use of membranes prompted the question as to the amount actually required. Inquiries directed to the Portland Cement Assn. and others failed to develop any firm opinion regarding the *critical* moisture

content. In 1947, T. C. Powers presented a paper at a Highway Research Board meeting in which he reported that concrete sealed against evaporation must initially contain more than about 0.5 g of water per g of cement to assure full hydration, since self-desiccation progressively reduces the space available for hydration products. This is about the same as the maximum water-cement ratio permitted in most specifications. Mr. Powers stated that membrane curing would not assure full hydration but, nevertheless, it should be adequate for concrete slabs in contact with the soil since the subgrade will provide a supplemental source of moisture.

The amount of water required for hydration and the mechanics of hydration are subjects of continuing research. However, based on present information there seems to be little reason to question the suitability of sealing membranes or of waterproof paper as curing methods for concrete pavement slabs when properly used under climatic conditions where soil moisture is available and daily temperature variations are not extreme.

Insulation

Theoretically, a covering of earth, straw, or mats has certain advantages over membranes for, in addition to maintaining a moisture supply, the cover acts as an insulating layer and minimizes detrimental volume changes induced by temperature variations. Under certain climatic conditions and for certain kinds of construction (concrete resurfacing, for example), consideration of such curing methods might be warranted on the premise that they should reduce tendencies toward shrinkage and curling and reduce the frequency of subsequent cracking.

CONCLUSIONS

It is believed that a more accurate and enlightening report of current practice for curing concrete pavement could have been prepared had there been time to write the Highway Departments and secure information from them, not only in regard to curing methods permitted but also in regard to the amount of concrete pavement constructed, the extent of use of the permitted curing methods and whether they give the desired results. Such a study is recommended.

Curing Structural Concrete

By H. J. GILKEY*

Ideal curing conditions for structural concrete differ in no way from those for similar mixtures in other adaptations. They are:

1. Continuously available free moisture at a sustained moderate temperature.
2. Freedom from stress that approaches closely the strength developed at time of its application.
3. Avoidance, upon termination of formal curing, of either rapid surface drying or of

*Member American Concrete Institute, Professor and Head, Dept. of Theoretical and Applied Mechanics, Iowa State College, Ames.

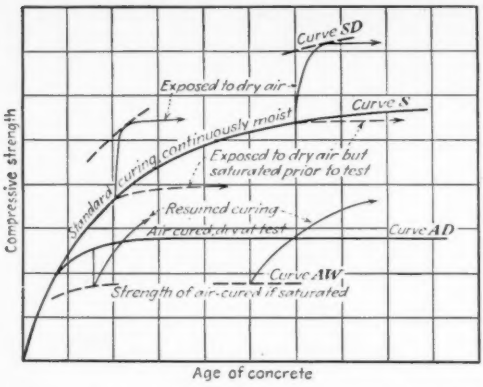


Fig. 1—Age vs strength for moist cured and air cured concrete

abrupt change of temperature (thermal shock) which would introduce damaging differential volume changes within the concrete.

4. Avoidance of exposure to freezing temperatures until curing has progressed far enough to partially empty the capillaries of their free moisture.

Fig. 1, evolved in the author's earliest researches on curing,[1,2,3] provides a reasonably accurate qualitative graph of the age-strength relationship for concrete from cement approximating Type I, cured at normal temperature but under various moisture conditions. Fig. 2 shows the corresponding effect of the curing temperature upon the rate of strength gain.

The difficulty in translating such evidence to job conditions stems not so much from lack of knowledge regarding the curing progress under known conditions of moisture and temperature as from inability to evaluate correctly the conditions that actually exist for a given concrete.

In building construction concrete the difficulty of insuring free moisture at all times during the formal curing period and of estimating correctly the probable effective mean temperature to be encountered by different members —some relatively massive, others thin—as well as maintaining a desired temperature, without sacrifice of the required available free moisture, creates ever-present problems requiring the exercise of a predictive judgment that in most instances can be little more than conscientious guess-work.

With the high surface-volume ratio (specific surface) of most building concrete, surface drying and temperature response is usually important and disturbing. While the members in structural concrete are rarely so massive that the accumulated heat of hydration, that characterizes mass concrete, is significant, there are likely to be appreciable temperature and moisture gradients from surfaces to interiors, which are bad both from continuity of curing progress and the corresponding differential volume changes.

SPECIAL PROBLEMS

Some special conditions surrounding structural concrete are:

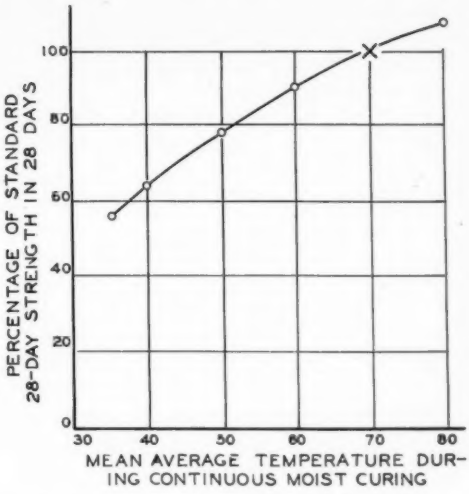


Fig. 2—Effect of temperature on the rate of curing of concrete

1. That formal curing is about the only opportunity the concrete will have to cure since the conditions of use are rarely such as to supply subsequent moisture.
2. In cold weather concreting, temperature invariably presents a major problem.
3. With both vertical and horizontal surfaces it is difficult to supply the needed moisture to all parts of all surfaces.
4. Early re-use of forms is generally of economic importance and complicates the problems of temperature, moisture retention and structural support. For beam-type members, especially, adequate support and formal curing are important for a relatively long time.

CURRENT STATUS

Because of the difficulty of insuring a continuously wet condition, whether by spraying, ponding, use of burlap—difficult to obtain, keep wet and high priced—supplying an effective drip down vertical surfaces and a general aversion by contractors to the use of paper coverings, there is an increasing tendency to use surface curing compounds. These require care to insure that they are applied at the earliest possible moment and in sufficiently heavy coatings. The surface of the concrete should be wet when the compound is applied. The compounds or membrane curing can be used on the floor slabs and unexposed frame members but exposed walls and other surfaces are likely to be darkened and mottled. Sometimes a fabric hose can be used along the tops of walls to provide a slow, steady leakage or drip. Leaving the forms on for a few days is considered one of the best practices for walls although this does not always insure effective moisture retention.

For cold weather concreting there have been important recent improvements in the methods of heating enclosures. The development of water-jacketed, oil-burning salamanders was a great improvement over the coke-burning salamander units, which in many states became inconvenient and expensive

as separate venting for each unit was required. The most recent advance is the use of oil or gasoline heaters with forced circulating air. These have great heat capacity and contractors claim that the more uniform heating of the enclosures adds materially to the comfort and efficiency of the workmen. Moreover, the transparent window-like plastics now available add greatly to the effectiveness of weather-proofing interior areas.

Another growing tendency is the use of insulating materials such as mineral wool, fiber glass and rigid insulating board. In many cases these materials, sometimes in combination with 2-in. wood forms for added underside protection, are used without artificial heat. Unless insulated, metal forms offer little temperature protection.

REDUCING THE PERIOD OF FORMAL CURING

By using a stronger concrete mixture (a richer Type I cement concrete, high-early strength (Type III) cement, or an accelerator such as calcium chloride used integrally, but not in excess of 2 percent) it is possible to reduce substantially the time during which formal curing is needed.

Data circulated by the Portland Cement Assn. indicate that Type III cement, with the same proportion and water-cement ratio normally used, will produce 3, 7 and 28-day strengths in about 1, 3 and 7 days. A two percent (by weight of cement) calcium chloride admixture will produce about the same result.

It is assumed that protection against freezing may be discontinued when the concrete has attained a compressive strength of 1500 psi. By this time the capillaries are believed to be sufficiently emptied to obviate freezing damage. Even soaking does not fully replenish water drawn from the capillaries in curing. Fig. 3 shows estimated lengths of time for different strengths of concrete,[4] including high-early strength on the right, to obtain 1500 psi.

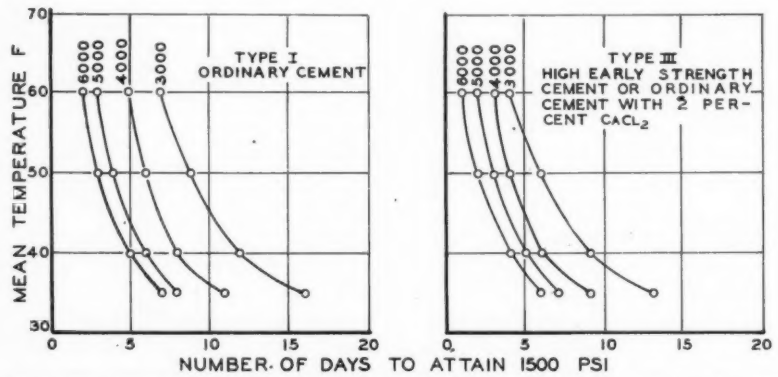


Fig. 3—Days required for concrete of different 28-day strengths to attain approximately 1500 psi when moist cured at various temperatures. Adapted from data supplied by the Cement and Concrete Association of London

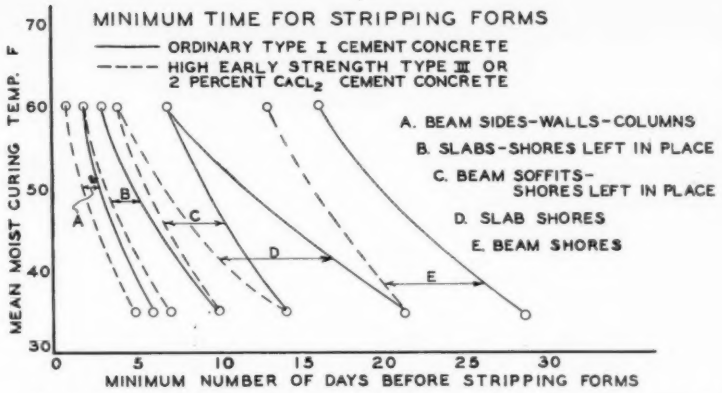


Fig. 4—Age at which forms may be removed from structural concrete, moist cured at various temperatures. Adapted from data supplied by the Cement and Concrete Association of London

Fig. 4 shows the minimum number of days for stripping form work for representative structural concrete,[4] presumably concrete in the 3000 to 4000 psi range. Fig. 4 compares the time requirements for high early strength *vs* ordinary concrete and the several classes of structural unit, for temperatures ranging from near freezing to about 60 F.

The data of Fig. 3 and 4 are not only themselves approximations, but the user is, as mentioned previously, confronted with the necessity of estimating the effective mean temperature. Such generalizations can be useful as an aid to the exercise of one's judgment, but where the stake is appreciable, the conservative engineer will use them cautiously or in conjunction with some form of check or control testing.

It is hoped that herein attention has been directed to the factors most in need of attention in the curing of structural concrete and that the data supplied may prove to be conservatively useful.

The author is especially grateful to Leo Corning of the Portland Cement Assn. for his aid in summarizing current U. S. practice.

REFERENCES

1. Gilkey, H. J., "The Effect of Varied Curing Conditions upon the Compressive Strength of Mortars and Concrete," ACI, *Proc.*, V. 22, 1926, p. 395.

2. Gilkey, H. J., *Engineering News-Record*, Oct. 14, 1937, V. 119.

3. Gilkey, H. J., *Concrete*, Jan. and May, 1941.

4. "Cold Weather Concreting," *Constructional Review*, Oct. 1951, V. 24:6, pp. 32-35.

Curing of Mass Concrete

By ROY W. CARLSON * and W. R. JOHNSON†

The fundamental principles involved in the curing of mass concrete are essentially the same as those for curing any other type of concrete. However, due to the thickness of the sections in mass concrete the effect of internal temperatures is a greater problem than in other types of concrete. Structures designated as mass concrete include large piers, abutments, dams and other massive concrete sections in which the least dimension is 2 ft or more. The effects of external curing probably do not penetrate to any great depth but, as no moisture loss due to drying takes place beyond this depth, the application of curing to the surface improves the quality of the concrete where it is most needed.

CONTRACTION AND EXPANSION

In addition to producing a good impermeable surface, resistant to weathering, erosion and cavitation, proper curing also aids in the reduction of surface cracking. Concrete expands upon heating or wetting and contracts with cooling or drying. All concrete has a tendency to crack if allowed to dry too rapidly before sufficient strength has been developed to resist this cracking. In mass concrete, cracks originating at the surface due to a lack of proper curing are good starting points for cracks which may extend more deeply, due to temperature changes within the mass caused by the heat of hydration in the cement. In a massive section the temperature of the concrete at the center may still be rising while near the face the temperature is falling. If the differential in temperature between the inside and outside occurs too rapidly and the concrete has not gained sufficient strength to resist this change, cracking may occur.

LIMITING HEAT IN MASS CONCRETE

There are several measures which can be taken to limit the heat in the concrete and hold the temperature differential to a minimum during the curing period. Cement with low heat of hydration can be used on any job, regardless of size. On large projects such as dams, where the minimum section may be approximately 50 ft in one direction and in excess of 100 ft in the other direction, cooling the concrete before placing, to as near 40 F as is practicable, aids materially in reducing the maximum temperature attained during the curing period. Embedded pipe cooling systems have also been used extensively for this purpose on the largest projects. Limiting the height of lifts, placed near the foundation, to a maximum of 2½ ft and the time between lifts to a minimum of 3 to 5 days are measures used to rid the concrete of excess heat during the early stages of curing. The use of steel

*Member American Concrete Institute, Consulting Engineer, Berkeley, Calif.
†Member American Concrete Institute, Senior Materials Engineer, Corps of Engineers, Portland District, Portland, Ore.

forms aids in the dissipation of heat from the surface at early ages and thus prevents building up too high a temperature at the face.

COLD WEATHER PROTECTION

During cold weather the first problem is protection of the concrete from freezing during the early curing period without allowing it to become too warm. Placing temperatures of 50 F are considered adequate. Due to the thickness of the sections involved advantage can be taken of the heat of hydration developed by the concrete to protect it against freezing. Unformed surfaces can be protected with tarpaulins when air temperatures are not lower than 20 F. By leaving the forms in place, formed surfaces can be protected down to 20 F, and even lower if insulation is provided by nailing a good insulating board to the studs or wales to provide a dead air space. At air temperatures much below 20 F, heat may need to be added, and it is advisable to measure corner temperatures for guidance. To prevent the concrete surfaces from drying, moisture should be provided along with the heat. When protection is being provided special attention should be given to corners and edges since these are the most vulnerable spots. In removing forms during cold weather, exposed faces must be protected until the curing period is completed, since a sudden temperature drop may cause cracking. When the form is raised, tarpaulins hung from the bottom of the form will temporarily protect the freshly exposed surfaces and slow down the cooling process.

On a dam having comparatively high transverse faces of blocks which are to be exposed for more than 30 days before being covered with concrete from the adjoining block, it has been found to be good practice to insulate these surfaces against sudden temperature changes to prevent cracks running parallel to the axis of the dam. This insulation can be provided by continuous water curing for the entire period of exposure or by covering with an insulating board after water curing for the normally required period.

While curing or sealing compounds are not generally used on mass concrete, special conditions sometimes exist which make water curing impracticable. Examples of this include curing in freezing weather, curing of walls adjacent to earth fills which are being compacted, and curing structures being built in a desert country where water is at a premium.

RECOMMENDED PRACTICE

Following is a summary of recommended curing practices for mass concrete based on the most recent report of Committee 612, Recommended Practice for Curing Concrete:

1. As a prerequisite to proper curing of mass concrete, arrangements should be made, when necessary, to insure a temperature for the mixing water and aggregates between 40 and 90 F. In work of sufficient importance and magnitude to justify the expense, the concrete, when necessary, should be cooled to between 40 and 60 F before placing. Concrete can be cooled after placing by the installation of a cooling system within the concrete mass to reduce the temperature to the extent required to maintain

a relatively uniform temperature, within the range for proper curing, throughout the whole mass. Use of a cement with a low heat of hydration is advisable.

2. Immediately following the cessation of placing operations for any reason, the exposed surface of the concrete should be kept continuously wet by spraying with water.

3. Immediately following removal of forms, the newly exposed surfaces shall be kept wet by spraying with water when air temperatures are in excess of 40 F.

4. Immediately following removal of forms, a colorless, impervious surface coating may be applied instead of water curing, if use of a water spray is impractical.

5. For outside surfaces of the concrete mass the water spray should be continued for at least three weeks for concrete made with low heat cements and for two weeks for concrete made with normal or modified cements.

6. For vertical construction joints between sections and for horizontal construction joints introduced through the suspension of placing operations, the water spray shall be continued until just prior to the preparation of these surfaces for the resumption of placing operations which will cover them with fresh concrete.

7. In the event of a long continued period between placing operations, the vertical joints shall be kept wet by spraying with water in the manner prescribed for outside surfaces in (5), and the horizontal joints by placing and maintaining upon the concrete for three weeks, a two-inch layer of wet sand.

8. In the preparation for resumption of casting operations, the surfaces to be covered by fresh concrete shall be kept wet for three days immediately preceding these operations.

9. Care should be taken either at the end of the curing period or upon removal of the forms to avoid sudden exposure to temperatures greatly different from those maintained during the curing period. Particular attention should be given to this feature in cold weather. Insofar as possible, exposure should be made in such manner as to allow the whole mass of concrete to gradually approach the temperature to which it is to be subjected immediately following the curing period.

10. Accelerators to increase rate of hydration of cement, and the use of cements with a rapid rate of hydration should be avoided in mass concrete because they increase the rate of heat evolution.

Curing Concrete in Canal Lining

By G. E. BURNETT*

It is general practice to cure concrete linings of irrigation canals by the application of a liquid membrane sealing compound. Engineers tried many types and brands of curing compounds and although some were worthless, others gave very satisfactory results. The economy of compound curing, particularly when used in desert areas, promptly became apparent when invariably chosen by contractors wherever it was permitted as an alternative to moist curing. This, coupled with other advantages, especially reduction of inspection and the prevention of sudden dry-out,† induced engineers to insist on curing concrete canal lining with sealing compound in preference to moist methods.

Sealing compounds have shown an evolution from the original black bituminous materials to clear compounds and finally to white or light-gray pigmented compounds which provide distinct advantages in minimizing absorp-

*Member American Concrete Institute, Materials Engineer, Bureau of Reclamation, Denver, Colo.

†Blanks, R. F., Meissner, H. S. and Tuthill, L. H., "Curing Concrete with Sealing Compounds," ACI JOURNAL, Apr. 1946, *Proc.* V. 42, p. 493.

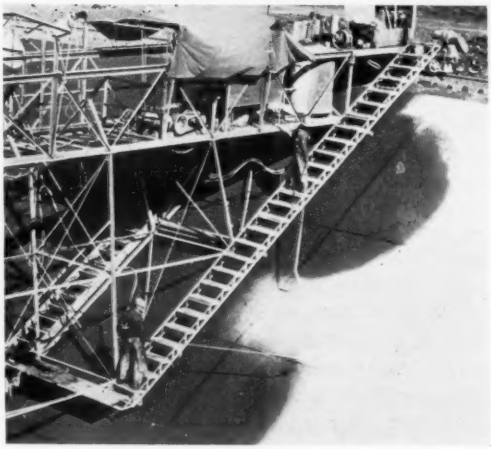


Fig. 1—Application of white pigmented sealing compound to concrete canal lining

tion of heat from sunlight, and in facilitating inspection of coverage. Maintaining cooler concrete tends to reduce expansion and contraction stresses which cause cracking. Also, the lower water-vapor pressure resulting within the concrete is conducive to a decreased rate of evaporation. Thus, of two compounds showing equal moisture sealing efficiency in a hot-room or curing-cabinet test, the one exhibiting the best heat reflection in sunlight will do the best sealing job on concrete canal lining.

Since 1946 white-pigmented compound has been used exclusively for curing concrete linings constructed by the Bureau of Reclamation with success and satisfaction.

APPLICATION OF SEALING COMPOUND

Sealing compound is customarily sprayed onto concrete lining (Fig. 1) promptly after finishing but not until free moisture has disappeared from the surface. The objective is to provide a firm base for the compound and at the same time to retain as much of the mixing water as possible. Recent laboratory tests indicate that optimum time for compound application is when the concrete has just attained a set.* Evaporation of mixing water prior to set appears to be beneficial by reducing water-cement ratio while the concrete is still plastic. Delaying application significantly beyond this point is damaging because excessive evaporation results in loss of mixing water needed for curing, and because a dry surface permits *striking in* of the compound which tends to soften the surface.

In practice, the optimum time for application will be reasonably approached by adhering generally to the established rules. Because of faster rate of setting, compound should be applied sooner in hot weather than in cold

*"Effect of Time of Application of Sealing Compound on the Quality of Concrete," Paint Laboratory Report No. P-50, Bureau of Reclamation, June 8, 1951.

weather. As a guide, the application should be made about the time the concrete has stiffened to the point where the surface will no longer respond properly to finishing operations.

Sealing compound should be applied in sufficient quantity to obtain a good seal. The necessary film thickness will vary depending upon the sealing efficiency of the compound and upon conditions affecting the drying of concrete. In Bureau work, a coverage of 150 sq ft per gallon of compound is required. This coverage, when employing a compound meeting Bureau specifications for sealing efficiency, will provide adequate curing for concrete linings even in hot arid regions. Under milder conditions the compound might be spread somewhat farther, but there is a danger that the compound may be too thin in spots unless care is taken to secure uniform coverage.

Uniformity of coverage is important, and herein lies one advantage of using white-pigmented compound. Only enough pigment is incorporated to produce a uniformly white appearance when applied to concrete at 150 sq ft per gallon. The contractor is not anxious to use more compound than specified and strives not to leave thin spots that will be revealed to the inspector, and require recoating. A black compound will obscure the surface even though applied much too thinly, and clear compound is often difficult to detect even though dye is added.

After sealing compound is applied, there is usually no need for further concern over curing. The compound is expected to remain intact and provide a tight seal for at least 28 days. If the seal is damaged in this period, immediate recoating is required. This seldom happens in curing canal linings because the work is essentially complete with the application of the compound.

The sealing-compound film usually weathers away gradually and uniformly by chalking. This is desirable in the interests of appearance. Occasionally peeling occurs, causing blotches, and this is believed to result from applying the compound too heavily or while the concrete is too wet.

Before white-pigmented sealing compound came into use, a day or two of preliminary moist curing was sometimes necessary in hot weather to obtain effective results with liquid curing compounds. This is not the case with the pigmented material, even in the arid Southwest.

MATERIAL REQUIREMENTS

Adequate specifications for curing compounds, and testing to see that the material furnished complies with the specifications, are essential to effective curing.

The Bureau uses a *performance* type specification. In addition to sealing efficiency, requirements are included for hiding power, reflectance, adhesion, weathering resistance, consistency, and absence of surface softening effect.

It is known that sealing compounds containing saponifiable substances, such as drying oils, have a tendency to react with and soften fresh concrete. For this reason, some engineers prefer nonsaponifiable, resin-base compounds.

Bureau experience, in the laboratory and on the job, is that a good wax-drying oil compound does not cause significant softening unless the concrete surface is overdry at the time of application and the compound strikes in.

AS AN AID IN WINTER CURING

When daytime temperatures are mild enough to permit placement of concrete but freezing occurs at night, the use of insulating mats may be the best means of keeping a concrete-lining job going.

On the Fort Sumner Project in New Mexico, the contractor used 1-in.-thick spun-glass blankets bound on both sides by a waterproof Sisalcraft paper for insulation. The blankets were laid over the day's run of lining before nightfall and left in place for 3 days, or longer when the weather was cold enough to stop work. After removal of the blankets, sealing compound was applied.

Temperature measurements proved the insulation to be effective in keeping the concrete well above freezing when air temperatures dropped to as low as 6 F at night.*

CONCLUSIONS

Apparent to anyone who has followed Bureau of Reclamation curing practices is that concrete canal linings on Bureau projects are being cured today essentially the same as five years ago. The application of white-pigmented sealing compound has proved an effective and economical curing method.

General Review of Curing Methods

By H. C VOLLMER†

Hydration of the constituents of cement continues to a limited extent throughout the life of portland cement concrete. This is evidenced by a continued gain in strength and hardness of concrete for many years after placing. However, practical curing procedures, both in the laboratory and in the field, usually terminate approximately with the attainment of the design strength of the concrete. The objective is to promote hydration and minimize volume changes during the critical period after initial set when the concrete has little structural strength to resist cracking.

The minimum protective period for slabs has been gradually reduced from a maximum of 28 days to the present accepted standard of three days. The improvement in design of concrete mixtures and the control of the mixing and placing of concrete have made this change practicable. Richer mixes have made strict adherence to early protection of concrete of primary

*Tuthill, L. H., Glover, R. E., Spencer, C. H. and Bierce, W. B., "Insulation for Protection of New Concrete in Winter," ACI JOURNAL, Nov. 1951, *Proc.* V. 48, p. 253.

†Formerly Research Engineer, National Bureau of Standards.

importance due to the increased water demand, rate of hydration, and increased tendency to shrink.

CURING AND ECONOMY

However, this inherent improvement in the quality of concrete has been responsible for frequent neglect of careful curing practice. The establishment and evaluation of acceptable curing procedures for portland cement concretes is too often influenced by economic considerations rather than the assurance of maximum curing results. The great demand for increased production rate, a longer season for concrete operations and possible emergency conditions wherein scarcity of materials exists are some of the more important factors to be considered. In the case of concrete pavements the production of as much as a mile per day can be expected. This rate of construction requires a *large* water supply for curing alone. These factors present a challenge to the concrete technologist; there is, however, considerable evidence that they can be successfully met.

FIELD AND LABORATORY CURING METHODS

Standard laboratory curing of portland cement concrete is essentially the continuous storage of specimens in the presence of free moisture at 70-75 F. This method applied for 28 days has long been recognized as a firm acceptance basis which insures thorough cement hydration. Early field curing methods simulated laboratory practice insofar as conditions would permit.

Specifications required thick coverings of damp sand, earth or straw, or ponding for 7, 14, and even 28 days. These methods although sound in their concept, were found to be impractical under conditions pertaining to large construction programs. Later burlap was used to retain the moisture and was found to have many advantages. An extension of the wet covering method is the use of the wet insulating blanket filled with cotton or similar absorptive material. In the group of wet coverings, mention should be made of the spray or sprinkling method. Fundamentally the action is the same.

The value of wet curing is generally accepted, particularly during the early period of the curing. In fact, where other curing methods are permitted, many specifications require overnight preliminary curing with wet burlap, after which the alternate curing methods may be used. This procedure materially reduces the outlay of the comparatively expensive wet-covering material and significantly improves the results obtained with alternate curing methods.

Impervious building paper sometimes reinforced with fibers may be used to prevent the moisture contained in the concrete from being lost by evaporation. Retention of wood forms serves to maintain moisture at the surface of structural concrete. Absorptive linings have been developed especially for this purpose. The type and condition of the surface upon which concrete is placed may have an important bearing on the curing of concrete. For instance, a dry porous soil base may remove moisture from highway con-

crete. Sprinkling of the soil base before placement of concrete is frequently specified to minimize the tendency of dry soil to remove moisture from the concrete. Waterproof paper is oftentimes placed on the soil base in slab construction.

Surface coatings of various types have long been advocated as a medium for curing of portland cement concrete. Tests of early materials, mainly of the bituminous and paraffin base type, indicated that although these methods had value as curing mediums they were not comparable in efficiency to wet covering. At the time of World War II interest in this type of curing was renewed because of the shortage of other materials. Low cost resin or varnish type liquids of high moisture retentivity were developed and made available. Fugitive dyes have been incorporated with the clear liquids to trace the area of coverage and a power spraying apparatus has been developed to insure uniform rate of application. The advantages of curing with surface coatings include lower cost, adaptability to vertical surfaces, extension of the drying period, convenience, and independence of external water supply.

CURING WITH CALCIUM CHLORIDE

Flake calcium chloride uniformly distributed over horizontal concrete surfaces is used as a curing medium in combination with preliminary wet coverings. The deliquescent nature of the calcium chloride under prevailing atmospheric conditions (relative humidity about 40 percent or above) will furnish an evaporation-resistant film of water on the surface of concrete, which will meet the hydration requirements of the cement. This method has the advantage of automatically furnishing additional moisture supply from the air and reduces the outlay of material and attention required by the continuous use of wet coverings for 3 days. The application of calcium chloride on concrete surfaces is recommended at the rate of 1½ lb per sq yd, after preliminary wet burlap covering is removed.

This method is one of the oldest used in highway construction and there are many thousands of miles of concrete roads in service today which were cured with this method. Laboratory test data indicate that this method produces early strengths equivalent to the use of continuous wet burlap for 3 days and in addition significantly increases the surface hardness of the concrete, which it is believed may be an important factor in durability.

All curing methods discussed previously dealt with the furnishing of sufficient moisture to insure adequate hydration. Inasmuch as hydration is the objective of curing, chemical means of promoting hydration fall in this category. Calcium chloride incorporated in the concrete mix in an amount of 2 percent by weight of the cement is used for curing concrete since it increases the early rate of hydration of the cement and strength development of concrete. Integral calcium chloride for curing concrete is used in combination with preliminary wet burlap. The marked increase in strength development of the concrete permits a reduction of the wet curing period to 24 hours, thereby resulting in a saving in wet covering materials. This

method is particularly applicable at low temperatures and in combination with the use of air-entraining and low heat cement where the normal rate of strength gain may be relatively slow. Earlier finishing is usually required and there is less latitude where construction delays may occur. It may be necessary to reduce the amount of calcium chloride at temperatures above 70 F.

Integral calcium chloride does not change the chemical reaction between cement and water except to reduce the pH of the solution which causes earlier precipitation of the gel-like products (the strength contributing products) in the cement-water solution. The total heat of hydration is not significantly changed, though the peak temperature due to heat of hydration occurs much earlier.

STEAM CURING

Steam curing applies particularly to concrete products and precast shapes. For many years the cinder block industry has stored the units in a saturated atmosphere at elevated temperatures ranging from 120 to 170 F for periods of approximately 24 hours or until the blocks developed sufficient strength to permit safe handling. In recent years considerable interest has been shown in high pressure steam curing. As a result some plants today use large autoclaves for pressure curing thus expediting hydration of the cement to the extent that the products may be shipped directly to the job without rehandling and storage.

CONCLUSION

One of the anomolies in concrete curing is that although specifications frequently refer to application of curing immediately after finishing, in practice the early application is too often delayed. There is a tendency to allow several hours between time of placing and initial curing regardless of atmospheric conditions during this time. The concrete is extremely vulnerable to attack during this period and particularly so at high temperature and low relative humidity.

Each of the curing methods described has advantages and disadvantages depending primarily on the type of construction, atmospheric conditions, availability of materials, and supervision required. There are ample test data available indicating that thorough application of the principles involved in these curing methods will measurable improve the more permanent characteristics of the concrete subjected to weathering. In any event, they provide the necessary factor of safety which is a fundamental engineering concept.

There still exists a divergence of opinion as to the method of evaluating curing efficiency. Correlation of field and laboratory data is difficult and the usual physical tests are not entirely reliable. Fundamental studies of cement hydration are being sponsored by Committee 612 and there is reason to believe that tangible results may be forthcoming.

Discussion of a symposium:

Curing of Concrete*

By M. SPINDEL and H. J. GILKEY

By M. SPINDEL†

From these papers it may be seen that curing of concrete is ranked in importance with design of the concrete mix and its making and placing. As a matter of fact, the method and length of curing for different types of structures and for the special circumstances of the job, especially with regard to weather conditions, should be considered in the design of the structure and the concrete mixes.

Although it has been generally known for a long time, it was well to emphasize, as did Mr. Timms, the warning that "too rapid drying of fresh concrete in the very early stages may result in crazing or cracking of the surface," that "under extremely unfavorable conditions severe cracking may occur," and that even "fine hair cracks will not be healed by subsequent curing." In discussing cold weather curing, Mr. Timms stressed the point that on such occasions "artificially heating the concrete introduces some danger of drying and interferes with the absorption of moisture" so "that concrete adjacent to these heating devices frequently shows signs of premature drying such as crazing cracks or a powdery condition of the surface." To this it may be added that crazing cracks or even the powdery condition may occur whenever the mixing water is dried out before the setting of the concrete, even if it is done only by wind and sunshine.

Less known to some concrete contractors is Mr. Timm's statement that "prolonged moist curing of air-entraining concrete without an intervening drying period will tend to fill the voids with water and reduce its potential durability" so that "in the case of air-entraining concrete for severe exposure a drying period should be employed following the initial moist curing period to ensure maximum durability." The tests on which that statement is based may be an important contribution to explaining the role of air entrained in concrete.

In several discussions of papers in the ACI JOURNAL and elsewhere, the writer was of the opinion that the higher resistance to weather and frost of air-entraining concrete was not due to the air *per se*, but to the part of air deliberately entrained by suitable admixtures, and that the beneficial effect

*ACI JOURNAL, May 1952, *Proc.* V. 48, p. 701. Disc. 48-46 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Research Engineer and Consultant, London, England.

was dependent, apart from the percentage of air which has to be strictly limited, on "the size, shape, and distribution of air pores in fresh and hardened concrete." In these discussions the writer also stressed the point that "those air pores which result from the evaporation of the mixing water which could not be bound chemically by the hardening of the cement, if exceeding certain limits, have the worst effect on the durability of mortar and concrete." He also suggested that "the examination and measurement of the various types of air voids has to be made along specially designed absorption tests, showing the absorption at various times and various conditions of immersion in water, to find the saturation coefficient."* These investigations would help in solving further the problems of curing.

With regard to the papers dealing with details of curing concrete for various types of structures, a few remarks only may be allowed.

Mr. Robinson refers to a paper by T. C. Powers in which the latter "reported that concrete sealed against evaporation must initially contain more than about 0.5 g of water per g of cement to assure full hydration." This appears to be in contradiction with the method of producing high quality concrete by using suitable admixtures for reducing the water-cement ratio, air entrainment, and vibration, resulting in water-cement ratios often below 0.5. Full hydration of cement during the usual curing period is not necessary and cannot be expected before many years. It seems that the right answer to this question was given by Mr. Robinson in saying that "the amount of water required for hydration and the mechanics of hydration are subjects of continuing research."

In the paper "Curing Structural Concrete," by H. J. Gilkey, ideal curing is concentrated excellently into four short points. Professor Gilkey realized the difficulties of fully complying with these points in practice and supplied useful information and advice based on his experience in the research laboratory and in the field, both from the theoretical and practical point of view.

With reference to point 3 dealing with the "rapid surface drying or abrupt change of temperature (thermal shock) which would introduce damaging differential volume changes within the concrete" and the further explanation of "appreciable temperature and moisture gradients from surface to interiors," the writer would like to point out that similar conditions cause difficulties also during the making of many other materials, such as iron, steel, and nonferrous metals. Depending on the speed of cooling during casting, and on some other circumstances, the structure near the surface is, as a rule, different from that of the interior. Often internal stresses occur which are the cause of cracks either during or immediately after cooling or at any later date, *i.e.*, when these internal stresses are combined with stresses due to fluctuating load, or changes of temperature which cause shrinkage or expansion with stresses exceeding the strengths of the materials.

*Discussion of "Effect of Entrained Air on Concrete Made with So-Called 'Sand-Gravel' Aggregates," ACI JOURNAL, Part 2, Dec. 1949, *Proc.* V. 45, p. 164-1.

With regard to concrete it is important to know that the effective shrinkage of mortar and concrete always begins at the start of setting and that the amount of shrinkage during setting may be even higher than later during hardening. The writer developed special methods of testing internal stresses in metals and also of testing shrinkage during setting of cement paste and mortar in 1916 and tested many hundreds of samples. Some of the results have been published in various European periodicals since 1925.*

The results of testing shrinkage during setting were the reason that when in 1916 the writer had to use his first high early strength portland cement for making a railway concrete arch bridge which had to be loaded two days after making the concrete arch, the probable shrinkage during setting was considered in the design of the size and position of the concrete block placed in situ. It appeared advisable that the forms should be released 24 hr after concreting to avoid shrinkage cracks in the special circumstances of the job in question.

In the writer's opinion the making and curing of concrete for every type of structure would benefit greatly from knowledge as to how shrinkage takes place during setting, if measured in various circumstances, such as at different temperature, different moisture content, with and without admixtures.

CLOSURE by H. J. GILKEY

In focusing his attention upon the volume change aspects of curing, Mr. Spindel aptly points out the close analogy between the differential volume change problems of concrete and those of such other materials as metal castings. One could also mention glass, especially in the casting of the large 100- or 200-in. lenses where differential shrinkages must be held down by cooling so slowly (over a period of months) that no portion of the casting ever has a temperature that differs appreciably from any other portion. As brought out in the paper by Messrs. Carlson and Johnson, "Curing of Mass Concrete," temperature gradients become especially important when the mass is large enough to prevent ready distribution and dissipation of the chemical heat of hydration of the cement. While in large dams, for example, the range of temperature change is small in comparison with that of a metal or glass casting, the dimensions of the concrete casting are so great that the distortional problems are similar.

In structural concrete the masses of members are rarely sufficient to make heat of hydration a dominant factor and the temperature aspect is almost solely that of safeguarding the concrete from environmental, rather than from internal self-induced temperature fluctuation. For the concrete technician to have in his consciousness the possibility of thermal shock, through abrupt changes in the weather, direct rays of the sun or ill-considered removal of forms or other protective coverings, is usually enough to insure his taking the appropriate and, in general, the relatively inexpensive precautions needed.

Zeitschrift des Oesterr. Ing. u. Arch. Vereins (Vienna), No. 11/12, 1925; *Zement* (Berlin), No. 14, 1936; "Question iii, Special Cements," Second Congress on Large Dams, Washington, D. C., 1936.

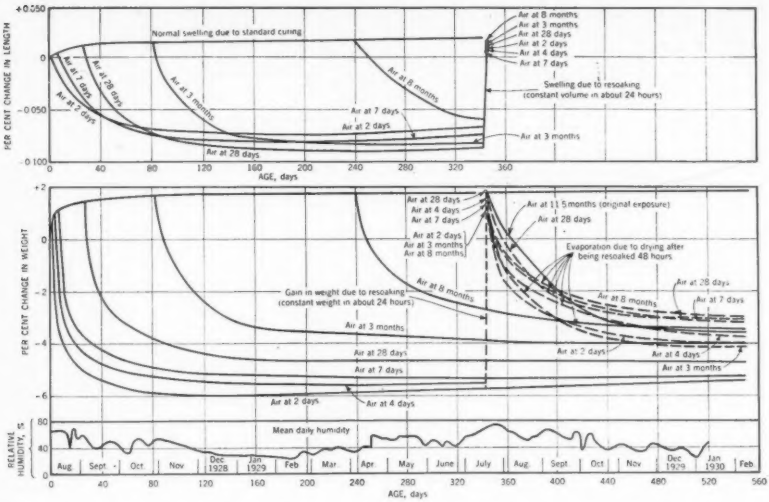

PER CENT CHANGE IN LENGTH
+0.050
0
-0.050
-0.100
Normal swelling due to standard curing
Air at 1 days
Air at 28 days
Air at 3 months
Air at 2 days
Air at 28 days
Air at 8 months
Air at 7 days
Air at 2 days
Air at 3 months
Air at 8 months
Air at 3 months
Air at 2 days
Air at 4 days
Air at 7 days
Swelling due to resoaking (constant volume in about 24 hours)
0 40 80 120 160 200 240 280 320 360
AGE, days
PER CENT CHANGE IN WEIGHT
+2
0
-2
-4
-6
Air at 28 days
Air at 4 days
Air at 7 days
Air at 2 days
Air at 3 months
Air at 8 months
Air at 11.5 months (original exposure)
Air at 28 days
Evaporation due to drying after being resoaked 48 hours
Air at 28 days
Air at 8 months
Air at 7 days
Air at 2 days
Air at 4 days
Air at 3 months
Gain in weight due to resoaking (constant weight in about 24 hours)
Air at 8 months
Air at 3 months
Air at 28 days
Air at 7 days
Air at 2 days
Air at 4 days
RELATIVE HUMIDITY, %
80
40
0
Mean daily humidity
Aug. Sept. Oct. Nov. Dec. 1928 Jan. 1929 Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. 1929 Jan. 1930 Feb.
0 40 80 120 160 200 240 280 320 360 400 440 480 520 560
AGE, days

Fig. A—Length- and weight-change diagrams for 3 x 3 x 40-in. beams exposed to dry air after various periods of moist curing

In most respects the mechanism of surface distortions from wetting and drying resemble environmental temperature effects but the moisture effects have their distinctive aspects. The swelling from wetting is relatively rapid in comparison with the shrinkage from drying, since dry (thirsty) concrete exerts a strong capillary pull, taking water at exposed surfaces much more rapidly than it releases water under normal atmospheric drying. Concrete is not distinctive in this regard; the same is true for brick, tile, and porous stone.

Even though surface-dry concrete has a fast affinity for water, the shrinkage from chilling (if the water is the cooler) can be detected before the swelling from wetting (filling small passages and soaking up of colloids) becomes measurable. All such actions and counter actions produce differential surface strains additional to those emanating from a restless exterior restrained by a relatively stable (as regards both moisture and temperature) interior.

Mr. Spindel, in his last paragraph, mentions the need for, "knowledge as to how shrinkage takes place during setting." From personal observations on this score, the author doubts that there is ever a "shrinkage" during setting (setting being used here, as Mr. Spindel has evidently used it, to mean "curing" or "continuing hydration").

Fig. A shows data first published as Fig. 164, p. 516, *Arch Dam Investigation.*[1] The two sets of curves for identical small beams show a slow increase in length closely paralleling the increases in weight due to added moisture for as long as the concrete was kept wet for continuing hydration.* Whenever the concrete

was exposed to the dry air of the laboratory there was drying shrinkage and loss in weight. These trends were always reversed upon reimmersion (except for the temporary dominance of temperature effect mentioned previously).

Other tests over the years have shown that as long as there is a continuing gain in strength from curing, there is a parallel continuing increase in weight. Evidently more moisture is being slowly annexed as long as hydration, with its corresponding gain in strength, proceeds.

Because of the high specific surface of the small beam-type specimens used, and the relative richness and fluidity of the mixture (virtually a mortar), the data of Fig. A are exaggerated in comparison with what should be expected from ordinary concretes, but the nature of the indications appear to be qualitatively definite and consistent.

The foregoing discussion relates to hydration or curing effects and takes no cognizance of the increase in strength which always accompanies air drying (being purely a drying phenomenon to be lost again upon re-soaking).[2]

While these matters are under discussion there is one other closely related factor that needs mention, *viz.*, drying shrinkage vs. plastic flow under sustained loading.

Mr. Spindel doubtless recalls the contention by Maney[3,4] that much of what has been considered to be plastic flow under sustained compressive loading is but the result of drying shrinkage. There unquestionably is a real plastic flow under sustained loading but where the specimens are kept continuously moist, the compressive shortening may not even equal the swelling from continued hydration. The mechanism of combination flow and shrinkage (the overloading of the core as successive rings of concrete undergo shrinkage shortening, thereby reducing the effective supporting area for the load as drying progresses inward across the loaded section), is ably expounded by Vogt.[5] As a practical measure of shrinkage (or swelling), one can, with little error, assume that the volume change from shrinkage (or plastic flow) will equal the volume of the moisture change that occurs during the same period. Where there is drying, there will be shrinkage; no drying, no shrinkage.

REFERENCES

1. Savage, John Lucian, Houk, Ivan E., Gilkey, H. J., and Vogt, Fredrik, *Arch Dam Investigation*, V. 2, May 1934, Engineering Foundation, New York, N. Y.

2. Gilkey, H. J., *Concrete*, Jan. and May, 1941.

3. Maney, G. A., and Gamet, M. B., discussion of "Flow of Concrete Under the Action of Sustained Loads," ACI JOURNAL, June 1931, *Proc.* V. 27, p. 1283.

4. Maney, G. A., and Lagaard, M. B., "Stress Increases in Compressive Steel Under Constant Load Caused by Shrinkage," ACI JOURNAL, June 1940, *Proc.* V. 36, p. 541 (with discussion in ACI JOURNAL, Sept. Supplement, 1940).

5. Vogt, Fredrik, "Effect of Shrinkage on Deformation of Concrete under Sustained Loads," *Det kgl Norske Videnskabers Selskabs Skrifter*, 1947; N.R. 1, 1 Kommisjon Hos F, Brus Bokhandel, Trondheim, Norway, 1949.

Durability*

SYNOPSIS

Report on conditions affecting durability of concrete structures, and discussion of design modifications and changes in construction and maintenance procedures to prolong their useful life.

Surveys of structures of various ages and an evaluation of factors affecting their durability. Brief report of research on testing and developing waterproofing materials.

A brief discussion of the factors that control durability of concrete pavements. Structural performance and durability are defined and their relationship to each other and to water-cement ratio brought out. The role of air entrainment in improving the durability of concrete pavements, particularly under the attack of chloride salts used for ice removal, is discussed and present thinking on the subject summarized.

The necessity of controlling volume change and providing impermeability in mass concrete hydraulic structures contributes to the problems of durability. Mix design, placement methods, use of admixtures and other measures to increase the serviceability of such work are considered.

The need for impermeable concrete is stressed as the primary requirement for lasting durability of mass hydraulic structures. Research in all phases of concrete mix and design is recommended as the answer to the need for a method of evaluating permeability in regard to its permanent effect on the durability of concrete thus establishing a realistic approach to specifying cement content of concrete mixtures in which strength is of secondary importance.

Railway Concrete

By C. B. PORTER†

Pioneers in the use of concrete for railroads, using their rule-of-thumb methods, built hundreds of portland cement concrete structures which are still giving good service. Many railroad structures, built in the early 1900's, remain in excellent condition as evidence of the good durability of well-made concrete. Despite this excellent record, concrete deterioration in railroad structures is much too frequent.

HISTORY OF RAILWAY CONCRETE

At the turn of the century very dry mixes were being used in concrete. However, between 1905 and 1910 began what might be referred to as the

*A group of papers presented at the ACI 48th annual convention, Cincinnati, Ohio, Feb. 26, 1952. Title No. 48-47 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Assistant Chief Engineer, Chesapeake and Ohio Railway, Richmond, Va.

wet mix era. This change was brought about to eliminate honeycomb and pitted surfaces due to dry mixes. Apparent improvement in workability and ease of placement were also noted. The 1904 "Specifications for Portland Cement Concrete"[1] required that, "The concrete shall be of such consistency that when dumped in place it will not require much tamping. It shall be spaded down and tamped sufficiently to level off, after which the water should rise freely to the surface." Text books of the period devoted some space to aggregate gradation but little was said regarding matters considered important today.

Soon after research in concrete was organized, the important and fundamental water-cement ratio law was discovered. This resulted in a gradual return to dry mix and more attention to aggregate grading.

Railroad contractors were skeptical of the new theory; their plants and equipment were designed and their personnel trained to mix and place relatively wet mixes. Many months passed before some of the old timers became convinced that well proportioned concrete with graded aggregate and uniform consistency resulted in economies to the contractor as well as increased strength and density.

Unfortunately, many railroad concrete engineers and masonry inspectors accepted the water-cement ratio strength relationship as the *cure-all* for concrete deterioration. However, destructive agents continued to work on concrete structures and it became apparent that chemical attack can result in deterioration even though the usual basic principles of making good concrete are observed.

SURVEY INDICATES HIGH MAINTENANCE COSTS

A questionnaire submitted to engineering officers of seventeen railroads, in July 1946, included the question, "Estimate roughly the amount of money your company has spent in the past 5 years to repair or replace disintegrated concrete." One railroad estimated an annual expenditure of $1,000,000. The average amount expended during the five-year period 1942–1946, inclusive, by the railroads submitting answers to this question, was $964,000.

On May 8, 1947, the Masonry Committee of the American Railway Engineering Association sent a questionnaire on concrete deterioration to chief engineering officers of member roads. Following are two of the questions asked, with the answers received.

1. What was the approximate amount of money spent by your railroad during 1946 for the repair of concrete structures to correct conditions directly or indirectly attributable to deterioration of the concrete? Thirteen answered none, 2 gave no reply, 1 didn't know, and 36 gave estimates which totaled $1,973,466. This represents an average cost of $54,819 for the 36 railroads responding, including some with few structures. Two of the larger railroads spent $347,000 and $250,000, respectively, on these repairs.

2. Have you observed any distinct difference in the rate of concrete deterioration in structures built previous to 1928 and those built since that time?

Six answered yes, 21 no, 1 questionable, 1 said that the latest concrete was poorest, and 23 said that concrete previous to 1928 was the poorest.

It must be admitted that the evidence brought to light by these questionnaires is not conclusive, yet there is a definite indication that repair and maintenance cost of deteriorated concrete in railroad structures is substantial. There is also evidence that concrete structures built by the railroads since 1928 are of better quality than concrete placed before that date although there are some concrete structures placed 40 or 50 years ago still in excellent condition.

DURABILITY

What are the railroads doing to provide more durable concrete structures? The basic durability characteristics of concrete are determined by the materials used, and the AREA, through its Masonry Committee, recently published a paper on this subject.[2] Its purpose was to summarize the physical characteristics of aggregates and their effect on the proportioning of concrete, and to present for the guidance of those engaged in the design of concrete mixtures, information covering the various properties of the more commonly used aggregates.

From the information presented in this report, it is apparent that aggregates have widely different characteristics which affect the proportioning of concrete mixtures. Aggregates from the same source may vary from day to day in average diameter of particles, absorption, specific gravity, density, moisture content and bulking and constant inspection and periodic testing must follow the original sampling and examination of aggregates if satisfactory concrete is to result.

EDUCATION AIDS IN PRODUCING DURABLE CONCRETE

A large part of the concrete placed on the railroads consists of relatively small jobs (culverts, station platforms, retaining walls and foundations) much of which is placed under the supervision of a masonry foreman, a bridge and building supervisor, or a young engineer from the division office. Many of these railroad employees have learned from experience and do not understand the fundamentals of proportioning, mixing, placing, and curing necessary to obtain good, durable concrete. In many cases their desire to increase their knowledge of concrete work is subdued by the technical terms and expressions used in text books or concrete manuals. The first reference to such terms as fineness modulus, impervious, monolithic, leaching, segregation, and water-cement ratio often result in the foreman or supervisor placing the text book in his closed file.

The Masonry Committee of AREA, recognizing the need for a simplified explanation of basic principles of concrete mix design and control, issued a report on this subject in 1949.[3] Before publication the text was rewritten several times in an effort to keep the explanations in simple nontechnical terms, always with the thought of stimulating the interest and understanding

of young inspectors and masonry foremen. This report is being used by construction and maintenance personnel of many railroads in the United States and Canada.

The Portland Cement Assn. offers the railroads short courses in concrete materials, design of mixes, handling and placing, and other problems involved in obtaining good concrete structures. Several railroads are taking advantage of these courses, and recently a course extending over three days was held on one railroad with forty men from the construction, roadway, bridge and building and signal departments attending. Included were top engineering officers, resident engineers, assistant engineers, masonry inspectors, and bridge and building supervisors. Short, practical courses of this kind enable those attending to keep abreast of advances made and equips them to make more durable and economical concrete.

CONCRETE DETERIORATION STUDIED

A realization that the railroad industry is incurring large expenditures in the repair and maintenance of concrete structures, and that pooling of available and prospective data on concrete durability will introduce improvements in materials and techniques that materially reduce repair and maintenance cost of such railroad structures, prompted the AREA Committee on Masonry and Waterproofing early in 1947 to initiate a program of research into concrete deterioration. This program is being carried out under the general direction of the research staff, Engineering Division, Association of American Railroads.

Four progress reports contain observations of the condition of 65 concrete railroad structures built between 1901 and 1929 in Indiana, Kentucky, Ohio, Pennsylvania and Tennessee. No conclusions have been reached by the sponsoring committees as the investigation is not yet complete. However, a few general observations may be of interest.

The first prerequisite for understanding the condition of any concrete structure is a thorough knowledge of the original characteristics of the concrete and of the materials used in its manufacture. The importance of such information was not generally appreciated when the majority of structures under investigation were built and the concrete records are not as complete as could be desired. However, data obtained by field investigation were supplemented by such records as were available and information given by railroad masonry inspectors, both active and retired. Concrete with best performance, probably representing the driest mix, was found in structures built in 1901 and 1905. Concrete mixes placed in later years appear to be somewhat wetter on the average, particularly in southern Ohio and southern Indiana.

RESEARCH REPORT LISTS CAUSES OF DETERIORATION

Of the 65 structures investigated to date, the probable cause of defects found in 16 must, for the time being, be classified as unknown. Observed

TABLE 1—OBSERVED CONDITIONS IN CONCRETE STRUCTURES

Defect and probable cause	No. of structures
Cracking and spalling due to excessively high water content of the mix	12
Open cracks, associated with strength loss, attributed to alkali-aggregate reaction	16
Popouts due to shale and fine grain soft limestone particles in coarse aggregate	6
Improper curing	2
Unfavorable reaction of large particles of slag	1
Good condition	12

defects and probable causes of disintegration in the remaining 49 structures fall generally into the classifications given in Table 1.

Problems connected with this subject are so diverse and involve so many variables that they cannot be solved by the unaided efforts of one or two investigators, however well qualified, over a period of two or three years. Study of the reports submitted to date, however, indicates good progress. It is believed that when this investigation is concluded, results will justify the expense involved and contribute materially toward knowledge of the cause of concrete deterioration.

WATERPROOFING RESEARCH PROGRAM

Several years ago AREA Committee 29 on Waterproofing recognized that concrete next to wet soil is subject to severe exposure and if not well made, may deteriorate with time. Therefore, it accepted an assignment to sponsor intensive tests of commercial waterproofing paints to develop data from which an acceptance test could be written. Such a test was considered necessary because of the large number of waterproofing paints on the market and the wide range of basic materials in their manufacture, such as vegetable oils, waxes, cement, iron filings and plastics.

Upon the request of Committee 29, a contract was made between the Joint Highway Research Project, Purdue University, and the Association of American Railroads to develop and conduct tests on waterproofing paints, of widely different basic types, to provide information pertaining to the durability and waterproofing characteristics of the individual paints.

The test program was developed by the Joint Highway Research Project, and the first tests began early in March, 1949 and were completed in March, 1951. This study was directed by Prof. K. B. Woods and J. B. Blackburn.

TEST PROGRAM

The study of waterproofings is in three parts. The first is an investigation of waterproofing paints, the second part will be performance tests of bituminous materials used for waterproofing and after the results of this work

are available, the third part, a study of bituminous membrane for waterproofing will be undertaken.

The purpose of the study on waterproofing paints was to develop test methods to evaluate paints and determine limiting values within which the paints would be beneficial. The work included tests on 104 different samples. All samples were subjected to one series of tests in which the paint was applied, in accordance with manufacturers recommendations, on 1 x 3 x 8-in. concrete specimens. After controlled curing these specimens were subjected first to an immersion test to determine the permeability of the coating, then to 150 hours in a Weatherometer, after which the permeability was again measured by immersion. Next, the specimens were subjected to 50 cycles of freezing and thawing, after which another immersion test was made. Subsequent to this, they were exposed on a rack to 90 days of outdoor weathering followed again by an immersion test.

Selected samples were tested for longer periods in the Weatherometer and more cycles of freezing and thawing. The selected samples were also applied to larger concrete specimens (3 x 4 x 16-in.) and exposed to natural weathering with one-half buried upright in the ground for 9 months, including one winter. Following this exposure, they were returned to the laboratory to be subjected to freezing and thawing ranging from 60 to 100 cycles, during which time deterioration of the concrete was measured by determination of the sonic modulus and absorption was determined by weighing. The tests on this selected group were made to obtain information for evaluating the tests on all samples. In addition, panels were painted on the wing wall of a railroad bridge, using selected samples, and these panels were examined periodically for 15 months.

TEST RESULTS

Preliminary test results show a wide range of performance between the most waterproof and the least waterproof paints, both as to waterproofing ability and to durability in weathering tests. Some of the painted specimens absorbed as much water during the immersion tests as the unpainted control beams; however, some were effective in slowing the ratio of absorption and others were only moderately effective.

Results obtained from these and earlier tests indicate that a product with which the buyer is unfamiliar should be subjected to an acceptance test. To quote from the report of Committee 29, ". . . the limiting conditions for an acceptance test for waterproofing paints can now be established, and that paints meeting these acceptance provisions will prove satisfactory as a waterproofing material." This committee is now working on a report covering an acceptance test and this should soon be available for publication.

CONCLUSIONS

In practically every case where concrete has failed, whether in railroad structures or elsewhere, there is a definite reason for such failure. Basically,

there is nothing wrong with concrete; the trouble seems to focus upon those who design, control, mix, place and cure it improperly.

REFERENCES

1. "Specifications for Portland Cement," *Proc.* AREA, V. 5, 1904, pp. 611-613.
2. "The Influence of Aggregate Characteristics on the Proportioning of Concrete," *Proc.* AREA, V. 51, 1950, pp. 372-380.
3. "Guide to Water-Cement Ratio Method of Making Concrete," *Proc.* AREA, V. 51, 1950, pp. 389-412.
4. Woods, K. B., Blackburn, J. B. and Lewis, D. W., "The Use of Waterproofing Paints for Minimizing Structural Concrete Deterioration," *Proc.* AREA, V. 51, 1950, pp. 876-891.
5. "Durability of Concrete," *Proc.* AREA, V. 42, 1941, pp. 335-352.
6. Terzaghi, Ruth D., "Concrete Deterioration," *Proc.* AREA, V. 49, 1948, pp. 634-639.

Railway Concrete Deterioration

By R. W. GILMORE*

Committees 8 and 29 of the American Railway Engineering Association sponsored a program of research on concrete deterioration. The investigation was made by Ruth D. Terzaghi for the Engineering Division of the American Association of Railroads, and B. L. Robinson, at that time concrete assistant of the Engineering Division Research Staff, American Association of Railroads.

They made four progress reports and reached the following conclusions. First and most general cause of deterioration was use of too much mixing water, second was the use of deleterious aggregates which caused popouts, third was excessive seepage of water from embankments, and fourth cause was volume changes due to shrinkage, that is, unequal expansion of concrete due either to volume changes of the aggregate because of temperature change, or to alkali reactions.

FOURTH PROGRESS REPORT

In connection with the Fourth Progress Report, Mrs. Terzaghi, Mr. Robinson, and the author made an inspection of 13 bridges in Southern Ohio and Indiana, most of them on the Baltimore and Ohio Railroad. Two of these structures were built in 1901, four in 1905 and 1906, one in 1910 and five between 1922 and 1927. Judging from appearances, structures built in 1901 and 1905 contained the highest strength concrete probably due to use of a drier mix. The later concrete appears to have been somewhat wetter, but only one example of very wet concrete was found.

A more thorough investigation of three of the B & O RR structures revealed that deterioration of the concrete was due to alkali reaction. This is the first time, presumedly, that this condition has been definitely established

*General Bridge Inspector, Baltimore & Ohio Railroad, Cincinnati, Ohio.

in the local area. A brief description of the structures and an investigation thereof follows.

No. 1, a concrete wall about 30 ft high and several hundred feet long, was built in 1901 and protects the railroad embankment from an artificial stream bed. At some spots near the base, this wall had cavities 3 ft deep caused by erosion. There was evidence of honeycomb, possibly due to pushing the concrete too far inside the form. There was no indication of laitance seams, indicating that a fairly dry mix was used. The wall has a ⅜-in. mortar facing which has conspicuous vertical cracks and numerous irregular fissures, ½ to 1 in. apart. The wall backs up to seams of wet shale and limestone, and is therefore wet a good portion of the year. Weeps have since been installed in an attempt to correct this condition.

The fine aggregate contained approximately 55 percent quartz, 25 percent limestone, 5 percent calcareous chert and siliceous limestone, 5 percent feldspar, about 10 percent chalcedonic chert and also some opal. The coarse aggregate is crushed limestone, 2 in. and down in size. Some is coarse grained stone and some a very fine-grained variety. Microscopic features indicate conclusively that a reaction has taken place between constituents of the cement and particles of chalcedony and of opaline chert contained in the fine aggregate. This reaction resulted in particles of both chalcedony and opal being partially or entirely converted to silica gel, which has a higher water content and occupies a larger volume. This causes a local splitting and disintegration of the concrete.

No. 2 is a steel railroad viaduct about 1000 ft long and 75 ft above the concrete pedestals. Corners of the towers are supported by concrete pedestals extending 12 ft out of the ground. Each pedestal is capped with a stone 24 in. thick. The concrete at the top is 4 x 4 ft, and 5 x 5 ft where it enters the ground giving considerable batter to the pedestal. These pedestals were evidently constructed of the same material as the wall and at the same time, 1901. They had, however, a mortar face 2 to 3 in. thick. It was Mrs. Terzaghi's opinion that the mortar facing represents a separate mix but the author believes it was made by someone inside the forms working mortar to the face to prevent honeycomb. The mortar face is very hard and exhibited no superficial defects such as scaling, popping, or crazing. However, there is present a network of cracks roughly rectangular and about 1 ft each way. Some of the horizontal cracks were set at construction joints. The number and size of the cracks increased near the bottom, and are practically absent in the top 2 or 3 ft. It is believed that this is due to the protection from drip by the stone coping. It was raining very hard during the first inspection and water actually spurted from the cracks when a train ran across the trestle. The pedestals were opened up for inspection and we found that the interior was soft and so fissured that it was difficult to obtain a fist-sized specimen. The sand used in this structure apparently was about the same as in the wall, but the coarse aggregate consisted of crushed limestone, chiefly of a fine-grained variety.

The results of this inspection indicate conclusively that alkali-aggregate reaction has taken place in the structure. The slight development of reaction rims on chalcedony particles in the mortar facing indicated that reaction was slight in this part of the structure because it is still in excellent condition, except for the presence of cracks due to the expansion of the concrete in the interior of the pier. Here the reaction was severe, as indicated by the thick reaction rims of silica gel on every chalcedony particle.

No. 3 is a double track railroad bridge built of reinforced concrete. It is large, consisting of three 70-ft arches and opening for a roadway approximately 15 ft high and 30 ft wide, and two extensive hollow, earth filled U-abutments. Drainage of the ballast on the top was inadequate, resulting in the structure being wet a great deal of the time. This structure was badly cracked, and ravelled out about 30 in. at one point, and in many places 5 or 6 in. Several years previous to the inspection it had been repaired by gunite and a drainage system installed on the deck. However, random cracks had penetrated the gunite. The fine aggregate was different from that used in the previous structures and contained about 60 percent limestone, 25 percent quartz, 10 percent chalcedonic chert and 5 percent siltstone. The coarse aggregate consisted of crushed calcitic and dolomitic limestone.

Examination of a thin section of a specimen taken from the south end of the underpass indicates that all but the larger particles of chalcedonic chert had been completely altered. In general, the peripheral portions of larger grains, and the entire cross section of smaller ones, have been replaced by carbonate. Since evidence from other structures indicates that similar particles escaped replacement by carbonates during the weathering process when they were in contact with carbonate-saturated water for many thousands of years, it is concluded that such replacement can take place only after gelatinization due to alkali attack. Hence the carbonate-impregnated parts of chalcedony particles are believed to represent reaction rims, which previously consisted of silica gel. If this conclusion is correct, the alkali-aggregate reaction has taken place in this structure on a scale sufficient to account for the observed deterioration.

CAUSES OF DETERIORATION

Deterioration in the other structures inspected may be classified as follows: cracking, scaling and spalling due to excessive high water content, popouts due to shale and fine-grained soft limestone particles in the coarse aggregate, and open cracks, associated with loss of strength, attributed to alkali-aggregate reaction. The first is readily identified and needs no further description. The popouts occurred in most of the bridges examined in which the coarse aggregate was gravel. This was demonstrated on several of these by breaking out a flat piece of shale laying parallel to the face of the concrete just under a thin coating of fine aggregate. Apparently if the face had not been spaded so carefully the failure might not have occurred. In two bridges the popouts were due to fine-grained limestone particles. Apparently there is

no reliable way of distinguishing this limestone from other varieties. The severest deterioration in the bridges examined and particularly on the structures described was due to alkali-aggregate reaction.

DETERIORATION DUE TO ALKALI-AGGREGATE REACTION

Alkali-aggregate reaction has two effects on concrete. One is the development of expansion cracks in the surface. For the trestle pedestals, the linear expansion of the interior exceeded that at the surface by approximately 0.3 percent. The second effect is a loss of as much as 40 percent of the original strength. The author believes that there had been a much greater loss in strength in the structures examined.

After concrete containing reactive aggregate and a high-alkali cement has been placed, there is no known way of preventing deterioration unless the concrete can be dried and all moisture excluded from it. Measures to improve the drainage will retard the reaction. Encasement of structures with new concrete or gunite may exclude sufficient moisture to prevent further deterioration, but it is doubtful if expansion can be wholly arrested. It is understood that some repairs are being made with provision for expansion of the new concrete included.

Surface waterproofing is not recommended; it is suggested that such treatment may retain moisture in the concrete as effectively as it excludes the moisture, and lead to an increase in the reaction.

RECOMMENDATIONS

The only completely reliable way to avoid deterioration due to alkali-aggregate reaction is to use aggregate which does not contain reactive constituents. In some regions, including the one under discussion, this would be feasible, except perhaps on jobs so small that even the moderate expenditure involved in the examination or testing of aggregate would not be justified.

In some areas, it may be more difficult to locate high quality aggregate either because of the scarcity of such aggregate or because of the existence of types of deleterious aggregate which can be detected only by time-consuming tests. If low-alkali cement is available, specifications should state that the alkali content of the cement shall not exceed 0.6 percent. The use of such cement may not entirely eliminate deterioration due to alkali-aggregate reaction, but it will undoubtedly reduce its incidence and its severity. Another method for the prevention of expansion due to alkali-aggregate reaction involves the substitution of one of certain pozzolans for about 20 percent of the cement. The AAR will continue their investigation of some of these structures and at the present time is trying to identify the source of some of the aggregates.

CONCLUSIONS

The question of popouts can be handled to some degree by the layman,

particularly in railroad work. A good deal of our crushed gravel is also used as track ballast. If the ballast muddies up quickly it is most probably due to the presence of particles of shale. These particles are the source of popouts and the use of gravel containing them should be avoided.

To avoid alkali reactive aggregate the sand and coarse aggregate should be thoroughly tested before acceptance for any large structure. The difficulty lies in the fact that failures due to this source may not show up for some years. In that period the pits and quarries may be mined out and most of the records as to the source of the materials discarded or lost.

It is not desired to leave the impression that alkali-aggregate reaction is the only cause of deterioration, or even the most common cause. It is emphasized here because the author was a member of the party making the field inspections which led to its discovery and identification.

Concrete Pavements Can Be Durable

By F. H. JACKSON*

The question implied in the title of this brief contribution to a very important subject has been bothering highway engineers for a long time. For example, surface scaling, particularly in the northern states was a problem long before it became the practice to use salt or calcium chloride for ice removal. Disintegration of the concrete, especially along joints and cracks following the formation of the so-called *D-lines* was also encountered. The problem has become far more serious since the use of chemicals for ice control, especially the use of the raw salts to obtain quick results. This question of scale resistance is undoubtedly the most serious aspect of the general problem of durability as applied to concrete pavements and therefore, ways and means of making pavement surfaces resistant to this attack are emphasized here.

STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

In any discussion of concrete pavement behavior it is necessary to distinguish between those properties of the concrete that are important from the standpoint of structural design and those which control resistance to repeated freezing and thawing, to the action of chemicals used for ice removal, to surface abrasion caused by tire chains and so on. It is to develop maximum resistance to these agencies that we continually strive to produce what we somewhat rather loosely term *durable* concrete. So far as structural design is concerned the only property of the concrete that is ordinarily considered important is strength, usually flexural strength. Some attention is being given to the effect of aggregate characteristics on volume change. Condition

*Member American Concrete Institute, Physical Research Engineer, Bureau of Public Roads.

surveys have shown that the cracking tendency of concrete pavements is influenced to a marked degree by aggregate type and considerable attention is being given by research workers to this problem. Some state highway departments recognize these differences by varying the joint spacing depending upon the type of aggregate used in the concrete. However, in most cases, strength is all that is considered by the designer.

It is well known that, for workable concrete with a given combination of materials, flexural as well as compressive strength is a function of the water-cement ratio. As long ago as 1928[1] I suggested a simple application of this principle to the design of concrete paving mixtures. So far as strength is concerned this procedure is just as applicable today as it was 20 years ago. Whether it will also insure durable concrete is another question.

AIR-ENTRAINING CONCRETE AND WATER-CEMENT RATIO

Until the introduction of air entrainment a little more than 10 years ago, it was generally assumed that changes in the W/C affected durability in about the same manner as they affected strength. Table 1 of the 1940 report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete is a good example of the application of this theory to specifications. In this table an upper limit of 6.0 gallons of water per sack of cement is specified for paving concrete subjected to severe exposure. This requirement, of course, makes it impossible to use the trial method of strength design in cases where the materials are of such a nature that concrete of the required strength can be produced with a W/C in excess of 6.0 gallons per sack. Such a limitation naturally penalizes high strength materials, particularly certain types of aggregates as well as cements having high early strength characteristics.

The advent of air entrainment changed this picture materially. In my opinion the improvement in durability that results from the use of the proper amount of entrained air in concrete is so great that variations in the W/C, at least within the usual working range, do not have the same significance as formerly. They are certainly not significant within the 0.5 gallon differentials set up in Table 1 of the Joint Committee Report. In spite of this there are those who would use Table 1 in specifications for air-entraining concrete without change. This would be a mistake because it would imply a far more precise knowledge of the relationship between the water-cement ratio of air-entraining concrete and durability than we now possess.

AIR-ENTRAINMENT PRIME FACTOR IN DURABILITY

A short time ago a series of tests was started by the Bureau of Public Roads to develop information along this line. Both plain and air-entraining concrete specimens with water contents ranging from 4½ to 9 gal. per sack are being subjected to alternate freezing and thawing in 3 types of cycle, a quick lab-

oratory freeze and thaw, 12 cycles per day, a slow laboratory freeze and thaw, 1 cycle per day, and outdoor exposure to freezing and thawing in sea water. No data are as yet available from the slow cycle freeze or the exposure in sea water. However, some rather interesting preliminary data from the rapid cycle test are available. For non-air-entraining concrete, the number of cycles required to reduce the sonic modulus of elasticity by 40 percent varied from 7 for the 9-gal. mix to 60 for the 4½-gal. mix, with the intermediate mixes showing values between these two extremes. In the case of the air-entraining mixes all specimens have so far been subjected to 620 cycles. In no case has the sonic modulus been reduced more than 11 percent, although in this group also such slight reductions in sonic E as have occurred are roughly proportional to the increase in W/C. Of far greater significance, however, is the remarkable improvement in durability of *all* of the mixes containing air. This is illustrated by comparing the richest non-air-entraining concrete with a W/C of 4½ gal. which lost 40 percent sonic E in 60 cycles with the leanest air-entraining concrete with a W/C of 9 gal. which has lost only 11 percent sonic E in 620 cycles. Furthermore, air-entraining concrete containing as high as 7½ gal. per sack, which is probably in excess of the highest W/C which would ever be used in air-entraining pavement concrete due to strength limitations, showed only a 5 percent loss in sonic E at 620 cycles. If these results are any measure at all of comparative field behavior, they surely indicate that in the design of air-entraining concrete paving mixes we can ignore the usual maximum of 6 gal. per sack, and use instead the W/C which will develop the required strength. As a matter of fact these and other investigations both in the laboratory and field as well as field experience with air-entraining concrete have about convinced me that we can ignore W/C, insofar as its effect on resistance to freezing and thawing is concerned, in most types of structural concrete, that is, concrete which must be designed for strength. It should be emphasized that the foregoing applies only to concrete which is exposed to simple freezing and thawing action, uncomplicated by sulfate attack, sea water or acid action or other special conditions where it might be necessary to use a W/C lower than would be required for the design strength.

This does not mean that, simply because we are using air-entraining concrete, we can ignore any of the basic principles that govern durability. It is just as important now as it ever was to use good cement and sound, durable well-graded aggregates and that well established construction practices be employed. These include the avoidance of overwet mixes, the use of vibration where feasible, and adequate curing.

CEMENT QUALITY

I am not one of those who feel that the quality of the cement is a major factor in controlling durability, at least within the range in quality and composition of the portland cements in general use today. It is true theoretically that variations in composition, fineness and other characteristics

of portland cement do influence durability. I have never, however, been able to find any convincing field verification of such trends. Certainly the results of the field study initiated about 10 years ago by the committee on The Long-Time Study of Cement Performance in Concrete indicate no such trends as yet. In my opinion the reason for this is that other factors such as quantity of cement, consistency, quality and grading of aggregates, mixing, placing and curing conditions far overshadow quality of cement in their effect upon durability.

AGGREGATE CHARACTERISTICS

Aggregate characteristics are of prime importance. They should be durable and well graded and if necessary, possess sufficient hardness to withstand the abrasive action of tire chains. This last applies particularly to heavy-traffic roads in the northern states where it is the practice to keep the pavement free of ice and snow. In general, more attention should be paid to the selection of aggregates than has been customary in the past. I recognize the economic limits to this suggestion. For example, we cannot ship high-grade aggregates a thousand miles or so to supply an area where only soft materials are locally available. However, with a given shipping area there are frequently possibilities of selection which are sometimes not used to advantage because of rather loosely drawn specifications.

FROST ACTION ON CONCRETE

Under conditions where concrete pavements are subjected to frost action uncomplicated by the use of chemicals for ice removal, or where such chemicals, if used, will be employed in moderate amounts mixed with sand or cinders, I feel that the proper utilization of the principle of air entrainment will come about as close to making concrete pavements durable as will be possible in this country for some time. Such conditions apply to many of our rural highways as well as to the streets of those cities where the practice of using the raw salts is, at least up to this writing, not permitted. The reader may ask to what extent a revision in our construction methods along the lines of current European practice, that is, drier concrete compacted by vibration or tamping, would help this situation. As indicated in the paper which I presented two years ago at the Boston regional meeting[2] I firmly believe that marked improvement not only in durability but in structural strength would be obtained by adopting these practices. However, I recognize the difficulty in putting them into effect in the United States, either now or in the immediate future. To achieve maximum benefits would mean completely redesigning and rebuilding most of the elements of our construction equipment and that would be totally impossible under present economic conditions. Therefore, under these conditions the only thing to do is to make the best use possible of our present construction equipment and the construction methods for which they were designed. It is believed that by adhering closely to the best current practices as detailed in state highway department specifications

and using intelligently the principle of air entrainment we can make concrete pavements which will be durable.

DETERIORATION CAUSED BY CHLORIDE SALTS

Tests by the Bureau[3] as well as field experience in New York State and elsewhere have indicated beyond much doubt that air contents within the usual specification limits will not necessarily protect concrete against the indiscriminate use of raw salt or calcium chloride for ice control. This indication is further verified by preliminary results of another series of tests in which specimens of air-entraining concrete having surface areas of 16 x 24 in. are being exposed to natural freezing followed by thawing with a surface application of 2.4 lb of flake calcium chloride per sq yd. At the end of 9 cycles all but 2 of the 27 specimens of air-entraining concrete are showing some evidence of surface scale, in some cases an appreciable amount. Corresponding specimens of non-air-entraining concrete had scaled virtually 100 percent after 3 cycles. This treatment is admittedly severe. However, it is no more severe than the treatment given many concrete pavements, particularly those on arterial highways within or close to our large northern cities. Moreover, these specimens have not been subjected to the action of traffic which undoubtedly serves to accelerate surface scaling once it has been started. With one exception, all of the 27 air-entraining specimens had air contents of 4.0 percent or higher, with 6 specimens running over 6.0 percent. Furthermore, before testing, all specimens were given 9 months air curing on the ground outdoors following 28 days moist curing. Therefore, the results cannot be ascribed to inadequate curing. A complete report of these tests will be released as soon as possible.

From the above it is evident that air contents within the usual range now specified, although of value in delaying salt action, cannot be depended upon to afford complete protection against the action of raw salts. However, there does seem to be a fairly definite relation between the amount of air and resistance to salt scaling. The tests reported in Timm's discussion of Tallamy's paper[3] indicated that for maximum coarse aggregate size of 1½ in. air contents of 6.0 percent or higher would give reasonably good protection. For this reason, the Bureau on Federal-Aid work has urged that the air content of concrete pavements which will be subjected to these severe conditions be held as closely as possible to 6 percent, with any adjustment necessary to maintain the design strength.

NEW CONCRETE PROTECTED WITH OIL

There is considerable evidence to indicate that a surface application of a light petroleum product, such as a light oil distillate will afford additional protection, particularly to new concrete. New York State originated this practice which has since been tried in other states. The theory underlying its use is based on the observed fact that the concrete immediately below the

oil stain which develops on heavily traveled concrete roads appears to suffer much less from salt action than other portions of the paved surface.

CONCLUSIONS

In conclusion, let us review very briefly some of the things which must be done to insure that concrete pavements will be durable.

1. *Cement*—Use a portland cement meeting the requirements of ASTM C 150, either Type I or Type II, with an air-entraining admixture conforming to ASTM Specification C 260-50T or an air-entraining portland cement meeting the requirements of ASTM C 175. Use ASTM Type III for high early strength.

2. *Aggregates*—Use the best of the economically available aggregates even though it may not be the cheapest. Low first cost does not always mean low ultimate cost. This may mean, at times, raising the minimum requirements shown in such national standards as ASTM C 33.

3. *Mix Design*—Using the materials which will be employed in the work, design the mix by the *W/C* trial method to give the specified flexural strength with a slump of from 2 to 3 in. for nonvibrated finishing or from 1 to 2 in. for vibrated finishing. Design for an air content of 5.0 ± 1 percent in cases where the maximum size of the coarse aggregate is ¾ in. or more. For small size coarse aggregate, such as sand-gravel, use 6 ± 1 percent. The foregoing is for cases where the pavement will be subjected to moderate amounts of salt mixed with an abrasive such as sand or cinders. In cases where it is anticipated that raw salts will be used for ice control increase these values by 2 percentage points.

4. *Construction Equipment*—See that all equipment is in good condition. Weigh batchers should be calibrated. Mixer blades should be inspected and replaced if badly worn. Finishing machines and spreaders should be in good operating condition and capable of handling the types of concrete specified under 3. Worn-out equipment so often seen on construction jobs should not be tolerated.

5. *Construction Methods*—Provide a sufficient number of qualified and responsible inspectors to see that all of the construction operations—proportioning, mixing, spreading, finishing and curing—are being carried out strictly in accordance with specification requirements. There is still far too wide a gap between theory and practice in concrete work and nowhere is this more evident than on inadequately inspected construction jobs.

Do these things and concrete pavements can be durable.

REFERENCES

1. "Design of Pavement Concrete by the Water-Cement Ratio Trial Method," *Public Roads*, Aug., 1928, V. 9, No. 6.

2. Jackson, F. H., "A Way to Better Pavement Concrete," ACI JOURNAL, Mar. 1950, *Proc.* V. 46, p. 489.

3. Timms, A. G., discussion of "Control of Concrete Scaling Caused by Chloride Salts" by Tallamy, B. D., ACI JOURNAL, Part 2, Dec. 1949, *Proc.* V. 45, p. 520-1.

Durable Concrete in Hydraulic Structures

By LEWIS H. TUTHILL*

Concrete can be durable, or at least much more durable than it has been in many hydraulic structures. The continued wetness of surfaces and satu-

*Member American Concrete Institute, Engineer, Bureau of Reclamation, Denver

ration of the interior of concrete in these structures make durable and serviceable performance more difficult to secure than in relatively dry structures. A wet condition fosters alkali-aggregate reaction and accelerates disintegration by freezing. Nevertheless, application of certain practices and materials in the past several years has produced structures which are so far giving good promise of durability definitely superior to that of many earlier structures.

In 1952 it borders on tiresome repetition to mention the great increases in durability now obtained as a matter of course as a result of the increased resistance to freezing provided by air entrainment, the increased resistance to alkali sulfates provided by the use of Type V cement, and the increased freedom from destructive alkali-aggregate reaction provided by the use of low-alkali cement and by opaline and other pozzolans. Although we have these great new foundations for expectancy of good durability their lack is not the only factor which can detract from it in hydraulic structures. Some further actions are often necessary.

DRAINAGE

Good drainage is imperative if good durability is to be obtained in hydraulic structures, especially where there is freezing or alkali sulfates, or both. Structures out of service should drain free of puddles. Tops of walls and all outdoor surfaces should have a positive slope that will quickly free itself of water. Weep holes should be prevented from seeping their effluent over the face of exposed concrete. Seepage or run-off water from higher ground should be prevented from flowing over the tops and faces of walls. Fig. 1 shows what can happen to the concrete when such water is allowed to flow over it, especially when it carries sulfates in a cold climate. The opposite wall from which the ground falls away, remained free of seepage water and is in relatively good condition.

COVER

Ample cover will greatly improve the durability of concrete pressure lines

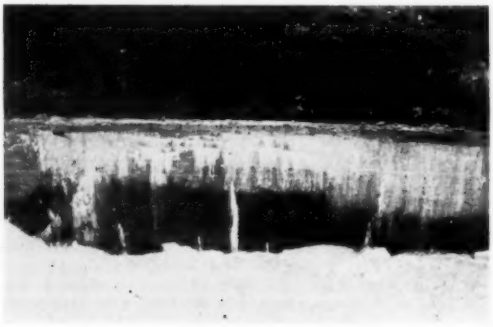


Fig. 1—Drainage must keep seepage or run-off water from flowing over the tops and faces of walls, especially where there is freezing or alkali sulfates, or both

such as cast-in-place concrete siphons in a canal system. Some older lines were either mounted on piers in part, or left with their upper portions exposed. Thus exposed, differential temperature changes caused cracks. With the concrete saturated and the cracks full of water due to operating pressure, the concrete is extremely vulnerable to freezing and thawing which occurs in frequent cycles in early spring and late fall.

JOINTS

Well made construction and contraction joints contribute materially to the durability of hydraulic structures because these structures usually contain or restrain water under considerable pressure. Fig. 2 and 3 show poor durability which was clearly augmented by freezing and thawing of seepage which came through poorly prepared horizontal construction joints. Thorough cleanup of each joint surface with wet sandblast and washing immediately before placing concrete, will prevent such leakage, especially if concrete placing is preceded by a thin layer of soft semifluid mortar well scrubbed into the clean wet joint surface. For good serviceability the same applies to horizontal joints that should be watertight in other structures as well as dams.

Ineffective and infrequent contraction joints result in leakage and leaching at such joints and at cracks which form between them. Rubber water stops at joints at which longitudinal reinforcing is discontinuous have been the most successful in maintaining water tightness. If they are spaced more than 25 feet apart in pressure conduits, intermediate cracks are likely to appear when the water temperature is low, and these often cause troublesome leakage. Lack of tight joints at proper intervals has often caused intermediate cracks and similar leakage problems in portions of powerhouse and pumping plants below the water surface.

Flow line structures such as flumes can be built at lowest cost with continuous reinforcing and no joints. With 0.4 percent or more of longitudinal steel, fine cracks, which soon become sealed, form at 2- to 5-ft intervals with-

Fig. 2 and 3—Well-made, water-tight construction joints would have prevented the seepage which, with freezing, was chief cause of disintegration of this surface concrete

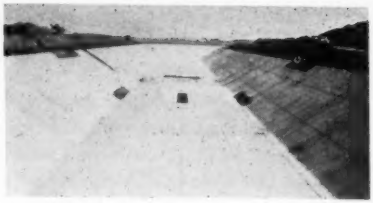


Fig. 4—Systematic grooving of unreinforced concrete canal lining keeps cracks in grooves where they can be readily sealed with mastic if silting does not seal them

out impairment of durability. Canal linings, which ordinarily require no reinforcing, should be grooved to ⅓ the thickness of the lining, at intervals ranging from 30 to 60 times the thickness of the lining, transversely as well as longitudinally (Fig. 4). The longer spacing is satisfactory on sandy subgrades where restraint is minimum. Shrinkage cracking is attracted to such grooves and, if these are not sealed by silting, the grooves make it much easier to seal the cracks with a suitable mastic material.

POZZOLANS

Leaching is reduced by use of pozzolan with the portland cement. As shown in Fig. 5, the surface of the concrete in many channels is slowly dissolved away by the action of pure or slightly acid mountain waters. Loosened sand particles rub off of such surfaces under the hand, showing that the loss of surface material is not by abrasion. Impermeable concrete, good

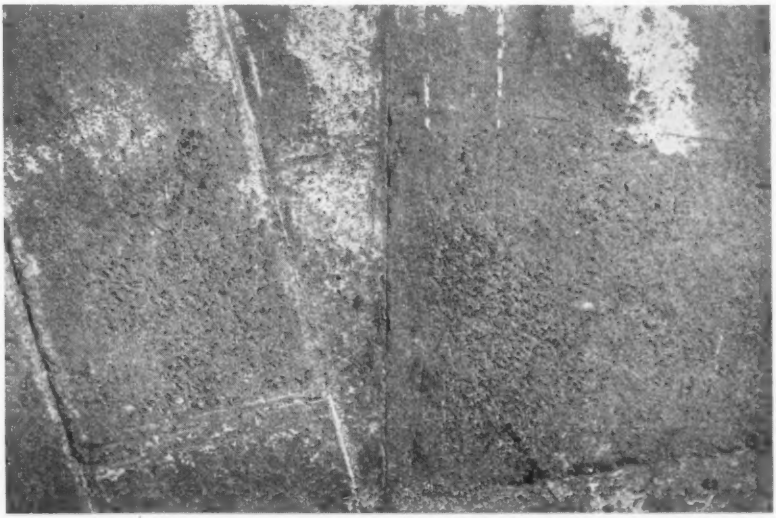


Fig. 5—Mineral free or slightly acid snow water can slowly dissolve surface mortar and expose small aggregate as in this siphon inlet. Good pozzolans reduce this by forming insoluble compounds with the lime

joints, and freedom from cracks prevent leaching within the concrete, but do not help the surface. This can be helped by using concrete of high quality and curing the surface well. Good pozzolans, used as a 20- to 30-percent replacement of cement, have been found to reduce the action of mild acids and the leaching of lime constituents from hardened concrete because pozzolans react with the lime to form insoluble silicates. Due to the moist environment of the concrete in hydraulic structures, conditions are most favorable for the continued and, therefore, the fullest ultimate development of the pozzolanic reactions. For this reason particularly good pozzolans are ideally suited to produce a superior concrete in hydraulic structures, provided the pozzolans are of a type which does not increase the initial water requirement of the mix by more than about 10 percent. Increase in water requirement means a proportionate increase in cement and pozzolan requirement for the same water-cement ratio. Also, large increases in water requirement may result in more shrinkage on drying than is likely to be offset by any greater extensibility of portland pozzolan cement concrete.

SURFACE ALIGNMENT

Good alignment of surfaces is essential if cavitation erosion is to be avoided where there is fast flowing water. This is a practical and direct solution to the problem of poor durability from this cause. Concrete cannot be made that will resist the destruction of cavitational forces, but it is comparatively easy during construction to align the surfaces over which the fast water will move so that there are no bulges and offsets. Fig. 6 illustrates the erosion produced by cavitation below an imperfection in alignment at a construction joint in a spillway tunnel.

SULFATE RESISTANCE

Factors other than Type V cement can be employed effectively to further increase resistance to sulfate corrosion. Another sack of Type V cement per

Fig. 6—Misalignment of surface (sight along rope) may cause serious cavitation erosion where water is fast flowing

cubic yard added to an ordinary mix containing 5½ or 6 sacks may be expected to increase the life of the concrete more than 50 percent. Contrary to some belief, an additional sack of Type II cement, unless it contains only 5½ percent of C_3A or less, will not produce sulfate resistance equivalent to that of a good Type V cement containing 4½ percent or less of C_3A (Fig. 7). Entrained air should be used because freezing often opens the way for sulfate attack and vice versa. After good curing, resistance will be greater if the concrete is dried for a time before exposure to sulfates. This is a particularly practical and inexpensive treatment for concrete pipe. Calcium chloride appreciably reduces resistance to sulfate attack and should not be used where such resistance is required. Where early strength is needed in connection with cold weather construction, another sack of Type V cement per cu yd in the mix will usually serve the purpose, and will greatly increase resistance as well.

TEMPERATURE CONTROL

Control of temperature differentials in mass concrete prevents extensive cracking in large dams. Dams are thus rendered more durable as well as more surely akin to the monoliths on which designs were based. This control is accomplished by placing the concrete at low temperatures initially, by cooling it with piped cold water after it is placed, or by employing low temperatures, air entrainment, low slump and vibration and pozzolan to keep the

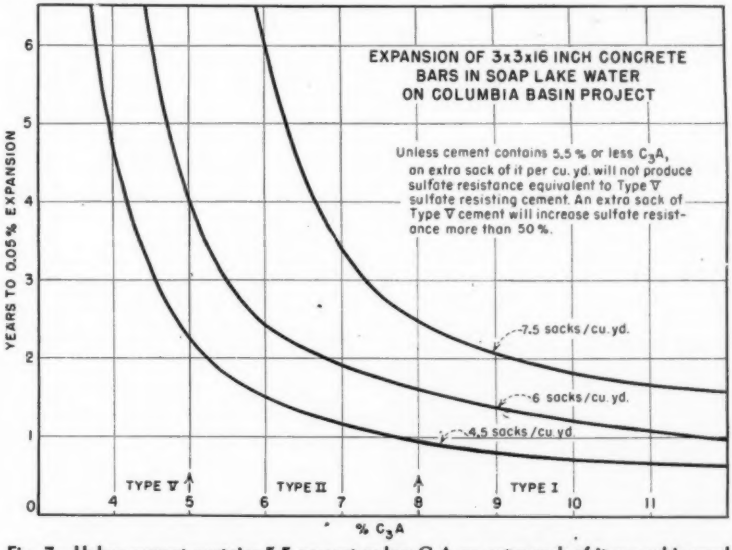


Fig. 7—Unless cement contains 5.5 percent or less C_3A an extra sack of it per cubic yard will not produce sulfate resistance equivalent to that of Type V sulfate-resisting cement containing 4½ percent or less of C_3A, but an extra sack of such Type V cement will increase sulfate resistance more than 50 percent

Fig. 8—Tops of walls can be made more durable by increasing cement content and by materially reducing slump to offset water gain

cement content and thus the temperature potential as low as possible. Effective combinations of these means are commonly used.

VIBRATION

Utilization of vibration to place sound concrete with minimum unit water content, and with a limited tendency to bleed, will improve durability. Low water content is not only a matter of low slump, but it is also obtained by using the largest practicable aggregate and the minimum practicable amount of sand. Vibration fully utilized makes these factors for reduction of water content available to a degree impossible otherwise. Wet concrete often results in water gain and decreased durability in upper portions of structures, such as the top of the flume wall shown in Fig. 8. Vibration during placement of the wasteway-chute lining shown in Fig. 9 would have produced a sound slab that would have remained in service for a long time instead of washing out and breaking up in a few years because of its weakness.

CONSTRUCTION PRACTICES

Good construction practices, other than vibration in placing the concrete, contribute to better durability of all structures. One of the most important is to prevent segregation of mortar of wetter consistency and to keep it from filling the corners, ends, and tops of walls. This material is usually weaker and more permeable on hardening and thus is more vulnerable to freezing. This can be prevented by reducing slump and then placing concrete first directly in corners and ends of a form and working toward the center. It can

Fig. 9—Concrete must be well consolidated by vibration if it is to be serviceable, particularly in a wasteway-chute lining

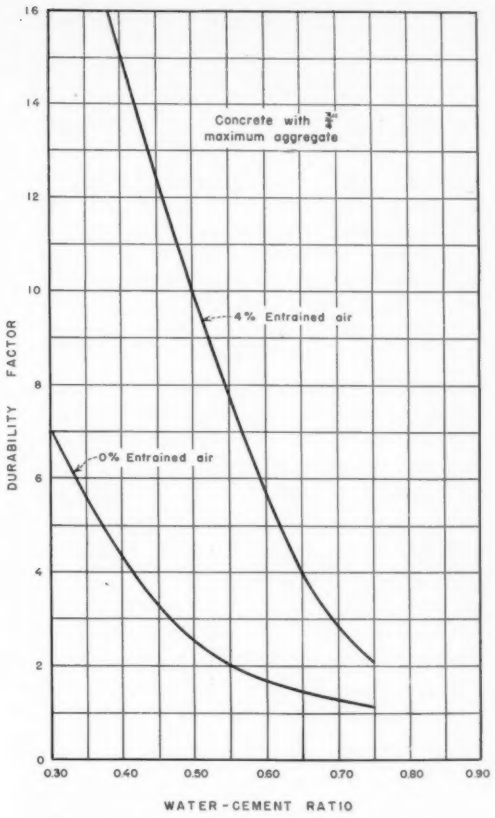


Fig. 10—At the same slump, durability increases with increased cement content and with lower water-cement ratios

be avoided at the top of lifts by reducing the slump to about 1 in. in the top 2 or 3 ft which are easily accessible for vibration. As is evident in Fig. 10, the lower W/C obtained by adding another sack of cement per cubic yard in the concrete will increase its resistance to freezing and thawing some 50 percent. This is inexpensive improvement in durability for those parts of a structure which are most vulnerable to freezing disintegration. Good curing always, and maintenance of temperatures at a minimum of 50 F for the first 3 days in freezing weather, will provide a maturity at the start that will aid future good durability.

With all these resources at hand it is safe to say today that concrete *can* be durable in hydraulic structures.

Durability of Hydraulic Structures

By BYRAM W. STEELE*

INTRODUCTION

It is not the intent of this paper to dwell on the failures or short-comings of concrete construction in the past but rather to generate enthusiasm for future possibilities. It is evident that the surface has not yet been scratched relative to the production of concrete that is durable over many years when subjected to severe weathering. This is especially true of water retaining structures in which there is a pronounced tendency toward the slow development of cracks, both at the surface and in the interior.

HYDRAULIC *vs* NONHYDRAULIC STRUCTURES

Comparison of concrete in hydraulic structures, such as dams and appurtenant works, with concrete in nonhydraulic structures, indicates that in too many cases water is finding its way through the concrete of the hydraulic structure while the nonhydraulic structure shows little evidence of water penetration into any part of the concrete mass beyond the surface film.

The distinction between hydraulic and nonhydraulic structures should be given more consideration in future design. Too much unwarranted criticism has been directed toward rigid concrete specifications for water retaining structures with the result that many have been led to believe that such rigid specifications are entirely unnecessary. An example is the following, quoted from an article in one of the leading technical publications:

> It may be stated that omitting refrigeration, permitting large differentials in height between blocks and rapid pours will result in cracks in the dam. In the language of the street, the answer is, 'So what?" Many mass concrete and masonry dams built in the last forty years do show cracks, but none of them has failed or even threatened failure because of such cracks.

In this regard at least one of the dams referred to in the above quotation is now in serious trouble because of such cracks and correction is going to be a difficult and costly job which may never be safe or satisfactory.

CEMENT SOLUBILITY

Since cement is many times more soluble than the average aggregate, wherever there is seepage of water into and through the interior of concrete there is bound to be deterioration through cement going into solution. Thus the infiltrating water weakens the bond and opens the door to a vicious cycle of more cement lost in solution and more loss of bond. Temperature variations at and near the surface play an important part in bond deterioration regardless of the presence or absence of noticeable moisture. Sooner or later this cycle of deterioration results in expensive repairs or replacements. Such is the history of infiltrating acid water in many areas in Norway and Sweden where the pH of raw reservoir and river water is predominantly low.

*Member American Concrete Institute, Consulting Engineer, Arlington, Va.

To counteract this basic weakness of porosity and volume change due to temperature variations, the Swedes and Norwegians changed their designs for hydraulic structures to reduce the tendency of the concrete to crack and admit seepage into and through the interior of the mass. This change in design largely eliminated the acid water infiltration and the deterioration that ruined many of their mass concrete structures.

There are many foreign countries where this acid water condition is a factor that must be recognized. In the United States such places are confined largely to areas along the Appalachian Mountains.

IMPERMEABILITY

Durable concrete in any climate must be impermeable. To be impermeable it must be devoid of cracks and the entire structure must be such as to resist all forms and forces of deterioration. It is regrettable that much of our current concrete mix design procedure and technique ignores the latter basic necessity.

With the coming of air entrainment impermeability in concrete received an important aid. The majority of users like air entrainment and it is universally applicable however, it is not a cure-all.

The latest new-comer in the field of concrete betterment is prestressing. It too is a step in the right direction where applicable. If concrete is kept under compression the development of many forms of cracks and progressive crack deterioration will be effectively discouraged.

Recent tests of permeability in concrete, although meager in extent compared with tests of strength and other properties of concrete, have demonstrated conclusively that the cement content of many concrete mixtures can be reduced materially and still produce sufficiently impermeable concrete for the interior of massive structures, and that continued hydration of the cement in concrete has the beneficial effect of appreciably reducing the permeability of the concrete with time. The latter fact is of special importance in hydraulic structures, such as dams and appurtenant works, because it assures a gradual decrease in infiltration of water under pressure over a long period.

FUTURE STUDIES

Research tests of any construction material are expensive and time consuming, but it is the writer's opinion that extensive research in concrete mixture design would pay big dividends in engineering, where so much important concrete receives so little attention compared with other construction materials. Such a research study would necessitate exhaustive tests of types and gradations of aggregates, types and gradations of cement, water-cement ratios, air entrainment, placing technique, curing methods, surface shrinkage and many other lesser factors. A half a bag of cement saved per cu yd of concrete in a few big dams would go a long way toward financing such a research study.

Excellent data on permeability were developed by Professor Raymond E. Davis during his recent contract with the Department of the Army. This work is being continued at the Corps of Engineers concrete research laboratory at Jackson, Miss., on a long time basis. In the writer's paper for the High Dam Conference at New Delhi, India, in 1951, there is given a rough comparison of the permeability of concrete specimens in the Davis Contract with the permeability of 10-in. cores drilled from the mass concrete in recent dams built by the Corps of Engineers, and also of 10-in. cores drilled from Arrowrock Dam built 37 years ago by the Bureau of Reclamation with two bags of portland cement per cu yd of concrete.

On the basis of the permeabilities obtained in the 1½ year tests of the Davis Contract specimens it would take several hundred years for water to penetrate the base of a dam 400 ft high. K_c values of 150 x 10^{-12} or less can be readily obtained at one year of age for mass concrete in the lower cement factor range used for the interior of dams.*

Theoretical impermeability in concrete is an impossibility because of the nature of the crystal structure of the vast majority of aggregates. However, practical impermeability is readily obtained wherever there is present a fair amount of that good old fashioned characteristic so well expressed in the saying, "where there's a will, there's a way."

Why not concentrate on perfecting an inexpensive and rapid method of evaluating permeability in regard to its permanent effect on the durability of concrete and thus establish a more rational yardstick for specifying the cement content of concrete mixtures in which strength is of secondary importance?

*K_c = cfs/ft², foot of head per foot.

Discussion of a symposium:

Durability*

By M. SPINDEL and BYRAM W. STEELE

By M. SPINDEL†

These papers contain the latest experience of prominent experts on the subject of durability and little new information with regard to the details of theoretical views and practical experience can be given in a short discussion.

In the papers dealing with hydraulic structures, especially that by Byram W. Steele, the need for impermeable concrete is stressed as the primary requirement for lasting durability of mass hydraulic structures, to which Mr. Steele expressly adds that "durable concrete in any climate must be impermeable." But in discussing impermeability (p. 749), the author says: "Recent tests . . . have demonstrated conclusively that the cement content of many concrete mixtures can be reduced materially and still produce sufficiently impermeable concrete for the interior of massive structures, and that continued hydration of the cement in concrete has the beneficial effect of appreciably reducing the permeability of the concrete with time."

This statement by the author is certainly true, but the question is whether or not the beneficial effect of reducing permeability of concrete with time has also the beneficial effect of durability, *i.e.*, resistance to water, weather and frost of the concrete on its surface, and even in the interior where the dissolving of the hardened cement by percolating water may surpass the influence of increased impermeability by the process of cement hydration during hardening. The following episode, which took place when the first large concrete dams in Europe were designed about 30 years ago, may illustrate what happened then and is still true with regard to permeability of concrete.

Large concrete dams made in Europe after the first world war were at first designed on the same principles as previous dams made of natural stones. The concrete dams, too, were supposed to be permeable to water, so that sufficient drainage with pipes, both in the foundations and throughout the dams, was considered necessary. Further, it was considered necessary to protect the whole surface of the dam which was exposed to water with a sufficiently thick and resistant watertight material which again had to be protected against damage from water and ice by a special wall, although it was not easy or reliable to do so.

*ACI JOURNAL, May 1952, *Proc.* V. 48, p. 725. Disc. 48-47 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 24, No. 4, Dec. 1952, Part 2, *Proceedings* V. 48.

†Research Engineer and Consultant, London, England.

The writer, then responsible for the design of concrete for two large dams in the Austrian Alps at an altitude of about 6060 ft, gave his opinion that the facing concrete, especially that exposed to water, had to be and could be made completely impermeable to water and also completely resistant to weather and frost, so that both drain pipes in the dam and its watertight covering and the special wall to protect the latter were not necessary. Concrete for the interior of the dam, which was protected from percolating water by the impermeable facing concrete was made with a materially reduced cement content. Without going into the many details of design and construction of these two dams, it may be said that the facing concrete on them withstood water, weather, and frost under the most severe conditions at that altitude and no water entered into or percolated through the dams.

Designers of large concrete dams in neighboring countries have come to similar conclusions. In one country, however, the engineers then made a mistake in principle in their conclusions from tests carried out by a special waterproofing commission. They came to the same conclusion with regard to the whole concrete of large dams as is mentioned by Mr. Steele with regard to the interior of massive structures. The fact of appreciably reducing the permeability of concrete with time led them to design even the facing concrete with a too low cement content and a high water-cement ratio to increase the workability of the concrete as it was placed. The result was that the concrete could not withstand water, weather, and frost, so that soon large parts of the surface had to be removed up to a depth of 3 ft 4 in. and replaced by hewn stones at high cost. Some experts on large dams then came to the conclusion that the damage was due to the use of modern portland cements, and others were of the opinion that facing with natural stones was necessary.

Many improvements on concrete have been made since, but it still is as said in the paper "Railway Concrete," by C. B. Porter: "In practically every case where concrete has failed, whether in railroad structures or elsewhere, there is a definite reason for such failure. Basically there is nothing wrong with concrete; the trouble seems to focus upon those who design, control, mix, place, and cure it improperly." To this it may be added that failures often occur also because the problems of cement and concrete are very complex and have not yet been solved completely.

In the writer's opinion concrete can be made more reliable than many types of natural stones that are used for structural purposes. The failure of concrete is often due to the deterioration of stone in the aggregates which cannot be protected sufficiently by the cement paste. Progress in concrete will continue with further research and especially with the use of suitable admixtures of which air-entraining and dispersion agents are the best known at present.

CLOSURE by BYRAM W. STEELE

In closing, the author wishes to point out that the "episode" in Mr. Spindel's discussion should be given thoughtful consideration. Many times laboratory studies and the results are given too much weight before being proved in actual structures. The factor of safety for the exterior mix of a concrete dam should be much higher than for the interior mix. Adequate research has not been accomplished in connection with the design and placement of concrete mixtures for the exterior surfaces of dams. Many dams have excellent concrete in the interior while the outside surface concrete is a disgrace. It is much more difficult to produce a durable, impermeable concrete surface on the downstream face of a dam than on the upstream face. The exterior surface must be a durable, impermeable, and crack-free coating to protect the inner less durable, less impermeable concrete.

Concrete mixtures for the exterior surface of a dam should have a low water-cement ratio, certainly not higher than 0.50 by weight. It is the author's opinion that 4 bags (376 lb) of portland cement per cu yd is the absolute minimum that should be used for the exterior surface of any dam. If the sand is not well-graded so that low water content and plenty of entrained air (6 percent) are easily obtained, four bags may not be sufficient to produce long life satisfactorily. Economy in the use of cement for a dam is best effected in the interior mixture since it is a relatively high percent of the total concrete yardage in the dam.

Title No. 48-48

Electric Heating of Concrete in Winter Construction*

By CHUZO ITAKURA†

SYNOPSIS

Describes an electrical method of heating concrete in cold weather which has been used in about 80 structures in northern Japan. The structures ranged from simple piers for wooden buildings to abutments and piers 82 ft high. The system, employing a network of electrodes, is applicable to plain or reinforced concrete. The cost of curing by this method varies from 7 to 10 percent of construction cost for plain concrete to 10 to 15 percent for reinforced concrete.

INTRODUCTION

Electric curing of concrete was first devised in Sweden about twenty years ago, and was applied to buildings and a subway in Moscow. Reports of this work were published in *Zement* and *Beton und Eisen* in 1932 to 1934, but nothing has appeared since.

In Hokkaido, Japan, in the early spring of 1947, field and laboratory studies were made on cold weather concreting with electric curing in connection with the U. S. Army Camp Crawford at Makomanai. During four years, including five winter seasons to the spring of 1951, data were obtained by the engineering school of Hokkaido University from about 80 structures and 7000 cu yd of plain and reinforced concrete. Among these structures are piers and foundations of buildings, pavements, linings of tunnels and inclined shafts, mass concrete of bridge abutments and piers about 82 ft high, the apron of a concrete dam, reinforced concrete pneumatic caissons 46 ft deep, wells of bridge piers, water tanks, reinforced concrete buildings and coal bins 68 ft high, and brick or stone masonry. In the specifications for cold weather concreting in Hokkaido and Tohoku (northern Honshu) districts, electric heating of concrete is now prescribed; especially at the various coal mines and for river works.

PURPOSE OF RAISING TEMPERATURE OF CONCRETE

In general, strength of concrete is not only dependent upon water-cement ratio, grading of aggregates, and mix proportion, but also upon curing temperatures, especially when the concrete is fresh. Fig. 1 shows the setting time of portland cement paste at various temperatures as determined by

*Presented at the ACI 48th annual convention, Cincinnati, Ohio, Feb. 27, 1952. Title No. 48-48 is a part of the copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952, *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Professor of Engineering, Hokkaido University, Japan.

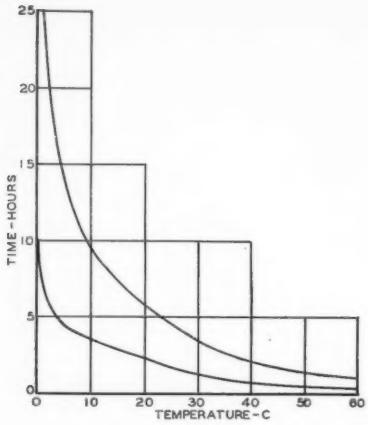


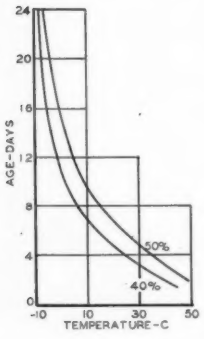
Fig. 1—Relation between temperature of cement paste and setting time. Upper curve—Finishing of setting. Lower curve—beginning of setting

laboratory experiment. When the temperature of the fresh concrete is low, as in cold weather concreting, initial set is delayed and the mix may freeze, causing frost heaving. The surface scales or the structure may collapse.

Even during hardening, if the concrete is exposed to the cold atmosphere while strength is still low, moisture retained in the concrete is frozen and the internal structure of the concrete is disrupted. The necessary compressive strength required in concrete to prevent such freezing damage is said to be 50 kg per sq cm (700 psi). Therefore, after concreting, forms must not be uncovered and concrete must be cured until the strength attains at least this value.

In addition, the part of the structure exposed to repeated freezing and thawing, as at the water line of structures in water, is destroyed by freezing. To prevent such damage, the minimum compressive strength must be 200 kg per sq cm (3000 psi) and absorption of aggregates mixed in the concrete must be lower than 2 percent by weight.

The relation between curing temperature and compressive strength of



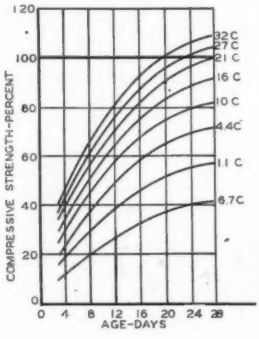


Fig. 2—Ratio of compressive strength of concrete at various curing temperatures and ages. 21 C (70 F) and 28 days taken as 100 percent. After McDaniel

concrete according to McDaniel is expressed in Fig. 2, and this can be referred to in determining the period of form covering and ability to carry loads. From Fig. 2, it is seen that strength can be gained early in ordinary portland cement concrete by means of high temperature curing.

This relation demonstrates the value of warming concrete, not only in winter concreting, but also in warmer weather, because of the reduction of the construction time. When using quick hardening cement, additional precautions may be necessary.

METHODS OF HEATING CONCRETE

Various methods have been used independently or simultaneously to prevent cooling and freezing and to allow generation of setting and hardening heat in concrete. Mixing heated aggregates and hot water, heating the atmosphere around the forms by stoves in a specially constructed enclosure, or by steaming are familiar means.

Heating concrete through a wooden form is inefficient because of the low heat conductivity of wood and concrete. In many instances, the main purpose is to raise the air temperature around the form to prevent cooling of the concrete. To transmit heat into the concrete itself, great quantities of fuel must be consumed and large scale equipment becomes necessary. Thus cold weather concreting has long been thought to be uneconomical. Moreover, fire or suffocation accidents are not uncommon in curing of concrete wells by this method. However, if heat can be conducted safely and cheaply into the interior of the concrete, cold weather concreting is practical and dependable, and hardening of concrete can be accelerated.

By electrical means heat is generated in the concrete itself by passing alternating current directly into the cement paste. Cement paste is decomposed by direct current or alternating current of 20 to 30 cycles, but alternating current of 50 to 60 cycles has no apparent bad effects on the concrete. Thus, the main effect of electrifying cement paste is heat, and any possible dangerous effects can be minimized by careful execution.

EFFECTS OF ELECTRIC CURING ON PROPERTIES OF CONCRETE

Strength

With respect to the modulus of rupture due to flexure and compressive strength of concrete, there are data available on differences between the strength of concrete cured by standard and electrical means. From experiments, modulus of rupture and compressive strength of concrete cured with electric current were found to be higher than that for ordinary standard curing, especially at early age. For older concrete, they are almost the same.

Bond stress between reinforcing bar and concrete

Experiments on pushing out a reinforcing bar embedded in concrete, show that when temperature difference between bar and concrete is less than 10 C (50 F) bond stress was increased in electrically cured concrete over that for standard curing. When the temperature difference was 25 or 26 C (77 or 79 F) with the temperature of the reinforcing bar as high as 80 or 90 C

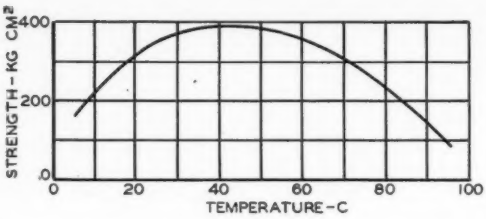


Fig. 3—Relation between setting temperature and compressive strength of cement paste

(176 or 194 F), the bond stress was about 80 and 34 percent, respectively. From these results, it is obvious that care must be taken in the electric curing of reinforced concrete, and that it is dangerous to heat reinforcing bars for a long time.

Maximum temperature of concrete

From experiments on the relation between temperature from beginning to finishing of setting and time of attaining maximum compressive strength of cement paste at constant temperatures of 2, 5, 10, and 15 C (36, 41, 50 and 59 F) in a constant temperature room, and at 20, 25, 30, 40, 45, 50, 60, 80 and 90 C (68, 77, 86, 95, 104, 113, 122, 140, 176 and 194 F) by electricity, Eq. (1) was obtained.

$$f_c' = 73.3 + 22.3\ T - 2.27\ T^{1.5} \quad \text{.....(1)}$$

Where

f_c' = compressive strength of cement paste, kg per sq cm; water-cement ratio 0.33, plasticity 5 to 6 mm, and age 7 days

T = constant temperature during setting, deg C.

The temperature corresponding to the maximum compressive strength can be obtained from Eq. (1).

When

T = 43 C (110 F)

Max f_c' = 400 kg per cm (5700 psi)

Values obtained by substitution in Eq. (1) are shown in Fig. 3. To allow for local irregularity, the temperature of concrete during electric curing must be thus kept below 40 C (104 F), and ordinarily below 30 C (86 F).

EQUIPMENT AND SUPPLIES

Equipment

All the apparatus is in common use and includes the following items.

Transformer—A single-phase transformer is convenient, but a three-phase one can be also used. An autotransformer adjustable from 10 or 20 volts to several hundred volts is desirable. If a transformer with this wide range of voltage is not available, an ordinary one may be used by adjusting terminals on the high tension side.

Switches—Two- or three-pole switches corresponding to the single- or three-phase source, with capacity of 100 to 200 amperes and several hundred volts, are necessary in numbers commensurate with the capacity of the transformer.

Voltmeter—Circular type for a switch board or a precision box type.

Ammeter—Circular type for a switch board or ammeter for line current. The latter is convenient to measure current in main wiring and in individual electrodes.

Wattmeter—An accumulating type is desirable.

Thermometers—A special electric resistance thermometer with terminals embedded in the concrete is best, but common mercury or alcohol bar thermometers may be used.

Lighting Apparatus—Flood lighting or electric torches are necessary to permit observing and measuring electric current and temperature of concrete at night.

Electrodes

If copper electrodes are used, contact resistance between electrodes and concrete increases and a small direct current may flow between copper and iron, which exists commonly in concrete. Sheet or strip iron, iron bar and iron wire are therefore used. For joining electrodes to branch wires, No. 8 or 10 BWG galvanized or plain wires are convenient, especially when electrodes are embedded in concrete. Even old wires, if loose rust is removed, can be used.

As main wiring, connecting transformer to electrodes through switches, both soft copper and hard types are serviceable in single and multiple circuits. Wires should be well insulated to prevent leakage of current on wet days or under moist conditions.

Branch wires connecting main wiring and electrodes are preferably insulated but occasionally uninsulated wiring may be used. Many soft thin wires are better than a few heavy ones, since they expedite electrode joining. They must be checked against the safe value of current corresponding to the amount of necessary electric power. Safe loads of wires and electrodes must be strictly observed, but if the time of electrifying is short, an overload of 20 to 30 percent may be allowable.

ELECTRODE SETTING AND ELECTRIFYING

Electrode setting

Electrodes may be placed (1) on the surface of the forms, (2) on the surface of the concrete, or (3) by embedding in the concrete.

Iron wire or sheet electrodes are fixed to the inside form surface ordinarily in a vertical position, with nails, staples or bolts (Fig. 4). By this method, electrodes can be accurately spaced and the wiring is not disturbed by concrete work. The disadvantage is that marks of electrodes remain on the surface of the concrete after removal of forms. In partial electrifying in which adjacent electrodes are electrified reciprocally anode and cathode, there may be much leakage of current along the form board when it is wet.

Electrodes may be set on the free surface of the concrete and stretched between side forms. When the distance between side forms is large, fixed placing of electrodes is difficult, so the electrodes are set after concreting is largely finished and are embedded in concrete a minimum of 5 cm (2 in.).

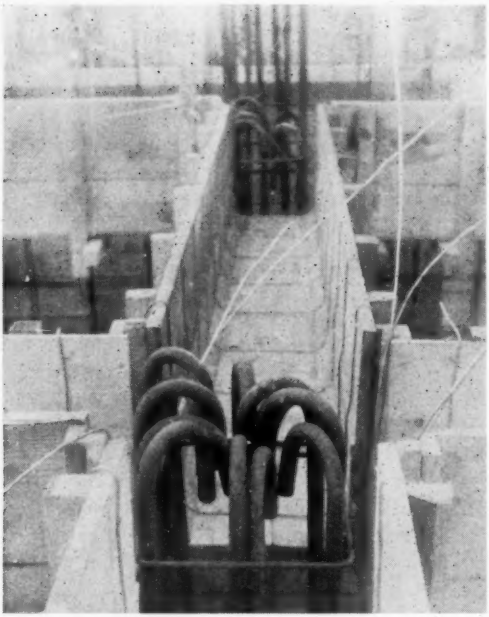


Fig. 4—Electrodes in place inside form

Electrodes must be embedded deep enough to prevent their being floated up by water bleeding to the surface of concrete, and to insure good contact with concrete. Completely embedded electrodes are in the form of rough nets. They can be easily stretched between forms, making use of tie wires or bolts (Fig. 5).

If the safe electrical load for the electrodes is exceeded, they overheat. Moreover, if an electrode is too long, current flowing in concrete near the feeder end is greater than that near the opposite end, so a uniform temperature rise in the concrete becomes impossible. Therefore, the length of electrodes must be limited.

Arrangement of apparatus

The general arrangement of electrical equipment for single-phase and three-phase electrical sources is shown schematically in Fig. 6. The difference between the two types of circuits is that two and three main wires are used for single and three-phase sources, respectively. For single-phase electricity, each electrode must be connected with a branch wire, coming alternately from the main wire. When a three-phase electric source is used, each electrode must be connected with a branch wire, in each series of 3 electrodes the fuse being connected with one pole, the second and third with the other pole, and so on in sequence, as shown in Fig. 6.

To join a branch wire and a sheet iron electrode, joint bolts are convenient.

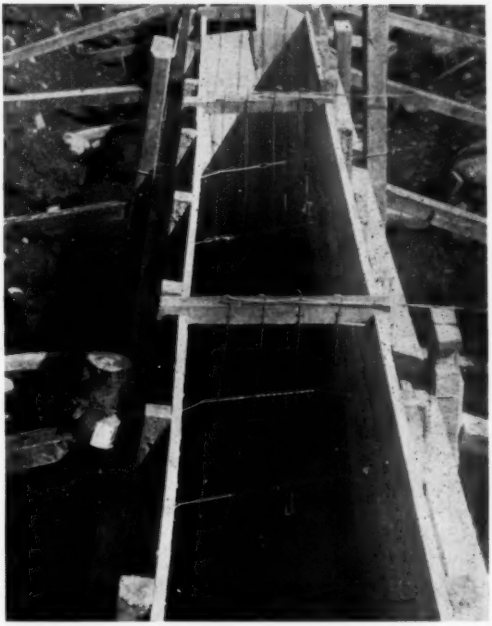


Fig. 5—Vertical electrodes attached to form ties

When there are many electrodes and the branch wires are thin and soft, the branch wire is wound around the electrode tightly at least five times to insure a good connection.

In many cases, electrode nets are placed horizontally or vertically before concreting. In the horizontal form electric current passes vertically through the concrete, between the upper and lower horizontal electrode nets. The lower nets are set before beginning to place concrete (Fig. 7) and the upper nets are set immediately upon completion of concreting or, if more convenient, in the course of concreting, so there are few possibilities of disturbing the concrete work. If the net electrodes are stretched too tight, there may be gaps left under them as the concrete sets, so it is wise to loosen the nets slightly a few hours after concreting is finished.

When successive days' concreting is not the same, the space between electrode nets may vary with resultant changes in electric power consumption and in the rise of the concrete temperature. Inversely, if power consumption and concrete temperature rise are to be constant, the height of a day's concreting is limited and placement must not be stopped even in case of accident. Moreover, if there is a difference between the upper and lower net areas, electric current and heat are concentrated in the smaller electrode net, so the ratio of the area covered by small and large nets must be kept within about 1:1.5.

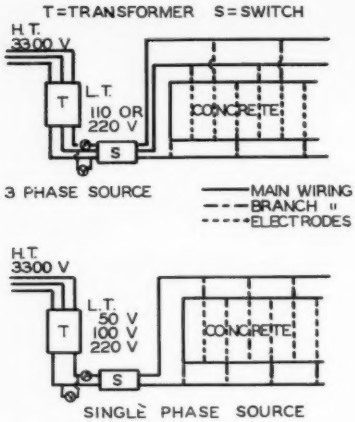


Fig. 6—General arrangement of electrical apparatus

In the vertical form (Fig. 8) electric current passes horizontally through the concrete, between vertical electrode nets embedded in concrete. The electrode nets can be set before concreting, so the work of embedding the electrodes in the concrete work is avoided. When the next pour is made on previously placed and already electrified concrete, the new concrete can be independently electrified by cutting off the electrodes at the surface of the old concrete. If branch wires are joined to the electrodes at their upper ends it is necessary to draw out the electrodes by perforating the form board. But, as the concrete is cast in comparted vertical sections, if the width is less than 1 m and the height of concrete is more than 2 m, placement is greatly hampered.

METHODS OF ELECTRIFYING

Section electrifying

The whole section of concrete is electrified so that the concrete temperature is raised uniformly with uniform heat generation during setting and hardening of concrete. This method is applicable to plain concrete and reinforced

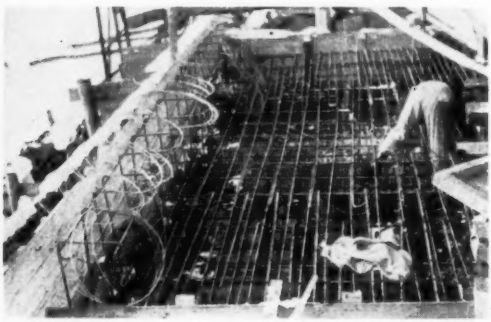
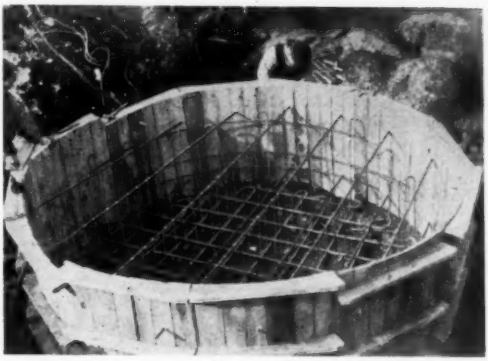


Fig. 7—Placing lower electrode nets prior to concreting

Fig. 8—Vertical electrode net in mass concrete



concrete, where the spacing between reinforcement is large or when there is a thick wall having an initial temperature below 10 C (50 F).

When the section is small, electrodes may be set on the form surface, but for large sections where electrode nets are embedded in the concrete, too much electrode material is required and the concrete work is also hampered. When electrodes are properly placed, there is no partial overheating, even if the switch is left on for 5 or 6 hours. Therefore, electric curing by this method can be done even by inexperienced workers.

Partial electrifying

In partial electrifying, part of a concrete section, usually only the exposed surface, is electrified. When the section measures less than 25 to 30 cm (10 to 12 in.), the results obtained are almost the same as by section electrifying. In a large section, the sudden rise in concrete temperature near the electrodes is pronounced, and when current is switched off, a sudden drop of temperature in that portion occurs also because of the electrodes' small enthalpy. In other words, in this method of electrifying, many linear heat sources are set at the cooling surface, and at the inner portion, generation of setting and hardening heat is gradual and delayed. If initial concrete temperature is more than 15 C (58.5 F), this method can be safely used.

There is a distinct possibility that concrete near electrodes may be hardened, while the concrete temperature between electrodes may lag considerably. Thus there may occur some initial strains unfavorable to the fresh concrete. Therefore, switches must be operated at 30- or 40-minute intervals. This method is commonly employed for heavily reinforced concrete.

FACTORS AFFECTING CONCRETE TEMPERATURE

In general, when metal conductors are electrified, the electric current becomes weaker with temperature rise. On the contrary, as ion conduction is involved in the case of cement paste, the electric resistance decreases with the rise in temperature and electric current increases. This correlation between electric current and temperature continues until setting of the

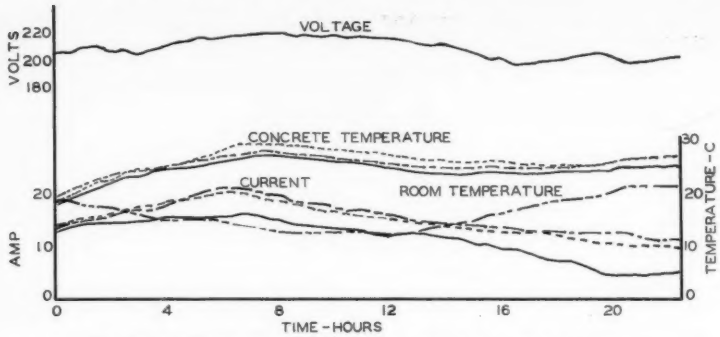


Fig. 9—Relation between voltage, current and concrete temperature

cement paste begins and the temperature reaches a maximum. As setting begins, electric resistance changes to that of a solid, and the volume of electric current decreases. The temperature drops with current, causing delay in the setting. If the voltage is constant, time-current and time-temperature curves are very gentle as in Fig. 9. If current and temperature change suddenly, the switch must be turned off and everything inspected minutely. In a large concrete mass, effects of setting temperature of the inner concrete become important, and temperature rises or remains constant a long time, even after the switch is off.

The relation between electric current and concrete temperature is

$$H = 0.24\, I^2\, R\, t = 0.24\, V\, I\, t$$

Where

H = quantity of heat generated in concrete by electric current, Joules
I = electric current flowing into concrete, amps
R = resistance, ohms
V = terminal voltage between electrodes of different poles, volts
t = time, seconds

On the other hand, the rise of concrete temperature becomes:

$$T = T_0 + \frac{1}{c\, m} H$$

And $$T - T_0 = \frac{0.24}{c\, m} V\, I\, t$$

Where

T, T_0 = final and initial temperature of concrete, deg C, respectively
c, m = specific heat and mass of concrete, g

Although c and m are strictly functions of time, t, let them remain constant with voltage and let t also be constant, for simplicity.

Then

$$T = T_0 + k\, I$$

Where $k = 0.24 \dfrac{1}{c\, m} V\, t$ = constant

That is to say, the temperature rise is proportional to quantity of electric current and this relation coincides with tests.

Voltage of source and spacing of electrodes are the most important factors. In a large massive section of a long, thin section of plain concrete, 200 volts is desirable since fewer materials are required. When the section is small, 100 volts or lower is preferred; especially in reinforced concrete.

When the terminal voltage is 200 or 100 volts, the relation between spacing of electrodes and density of electric current is expressed experimentally by

$$\left.\begin{array}{ll} \text{For 200 V;} & I_i = 2.02/x^{0.6},\quad I_{max} = 3.01/x^{0.6} \\ \text{For 100 V;} & I_i = 4.45/x,\quad\ I_{max} = 5.56/x \end{array}\right\} \quad \ldots\ldots(2)$$

Where, I_i, I_{max} = initial and maximum electric current density, flowing through concrete section perpendicular to the direction of current flow, amp per 100 sq cm

x = spacing between dissimilar electrodes, cm

Mix proportions of concrete = 1:2:4 by weight

Slump = 15 to 18 cm (6 to 7 in.)

Initial temperature of concrete = 15 to 19 C (59 to 66 F)

These relations are shown in Fig. 9.

From Eq. (2), the maximum quantity of electric current and the maximum electric power consumed per cu m of concrete are reduced to:

$$\left.\begin{array}{ll} \text{For 200 V;} & P_i = 4.04 \times 10^3/x^{1.6},\ P_{max} = 6.02 \times 10^3/x^{1.6} \\ \text{For 100 V;} & P_i = 4.45 \times 10^3/x^2,\ \ P_{max} = 5.56 \times 10^3/x^2 \end{array}\right\} \quad \ldots\ldots(3)$$

Where P_i, P_{max} = initial and maximum electric power consumed, KW per cu m of concrete

P_i and P_{max} are expressed in watts, and divided by voltage give values of electric current. These results are shown in Fig. 10.

Ordinarily these values can be exceeded, except in cases of excessive current leakage, and coincide with actual data within a maximum error of 10 to 20 percent. When the concrete mixture is dry and slump is below 5 cm (2 in.), the current density decreases about 20 percent.

The relation between electric current density and the maximum concrete temperature is expressed in Eq. (4).

$$\left.\begin{array}{lll} T_{max} = 13.8 - 93.18\, I_{max} & & \\ \quad = 13.8 - 280.35 / x^{0.6} & \text{(For 200 V)} & \\ \quad = 13.8 - 518.2 / x & \text{(For 100 V)} & \end{array}\right\} \quad \ldots\ldots(4)$$

Where T_{max} = the maximum temperature of concrete, deg C

The spacings of dissimilar electrodes, ordinarily used, are:

x = 100 to 130 cm (3.3 to 4.3 ft) for 200 V

x = 30 to 50 cm (0.9 to 1.5 ft) for 100 V

Quantity of water, cement, and mix proportions

The more cement and water in the concrete mix, the larger the quantity of electric current required. In other words, rich wet concrete consumes more current and its temperature rise is more rapid than for a leaner dry mix.

When aggregates segregate, the quantity of electric current flowing in the parts composed principally of aggregates and those principally of cement paste differs greatly. Therefore, concrete to be electrified must be properly mixed. Quick hardening cement paste consumes much more current than common portland cement paste.

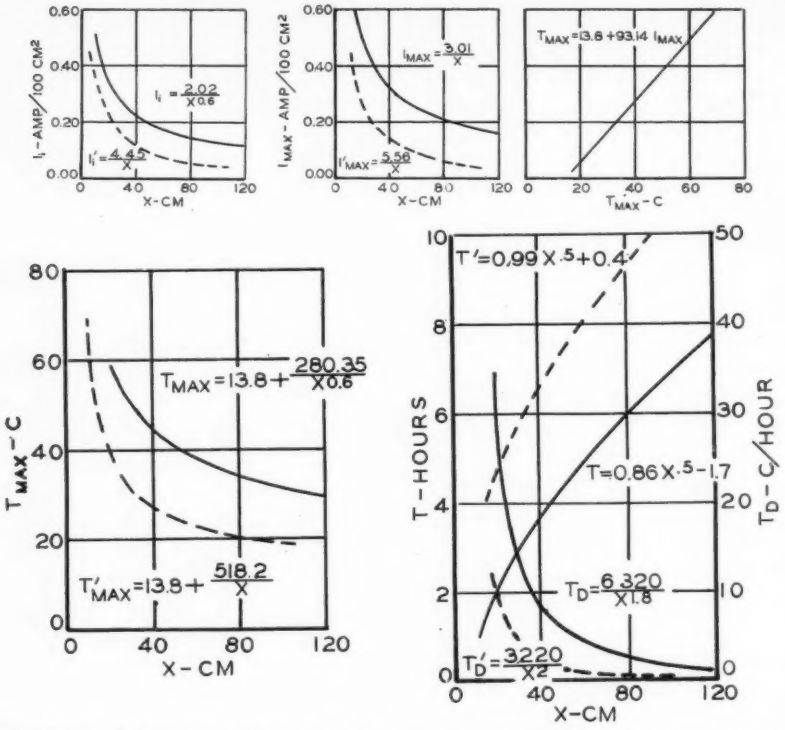


Fig. 10—Effect of electrode spacing, initial and maximum current density on maximum concrete temperature, ratio of temperature rise and hours to maximum concrete temperature. Mix proportions 1:2:4 by weight, W/C = 0.75, slump 15 to 18 cm. Specimens 10 x 10 x x cm with No. 8 BWG galvanized electrodes. Electrified continuously for 24 hours at room temperature of 12.7 to 21.0 C (54.9 to 69.8 F)

Size of aggregates

Since aggregates are insulators, there is no heat generated in them, and they are heated by the cement paste around them. The coarser and cooler the coarse aggregates, the more slowly they are heated. This condition is particularly noticeable in boulder concrete where occasionally a thermometer inserted in the mortar will show a high temperature, while one near a boulder or touching it will show a very low value. In such a case, the switch must be turned off for a time to allow equalization of temperature between boulder and cement paste.

Size and form of concrete section and climate

When temperature of fresh concrete or the atmosphere is low, or winds are strong and cooling is severe, the electric current flows poorly and temperature rise is slow. Therefore, the temperature rise of concrete of small section and large cooling area is more delayed than that of larger section with smaller

cooling area. Electric power consumption is less and longer electrifying time is needed.

Mixing water and concrete admixtures

If an electrolyte is dissolved in the mixing water or an admixture is used in the concrete, current consumption and concrete temperature rise are changed materially. Concrete mixed with salt water has small electric resistance, so that the quantity of current flowing in the concrete is great and temperature rise is slow, requiring extensive changes in the design of electric curing. Experimental data on concrete mixed with sea water, containing 3.75 percent salt by weight, substituted in Eq. (2) and (4) gives the following:

$$\left.\begin{aligned} I_i &= 104/x,\ I_{max} = 120.5/x \\ T_{max} &= 14.1 - 42.08\, I_{max} \\ &= 14.1 - 50{,}706/x \end{aligned}\right\} \quad \ldots\ldots (5)$$

where the voltage of the electric source is 200 V, thus special care is necessary in concrete work in coal or metal mines and along the sea coast. When lime is added to cement as an admixture, the same phenomenon results. Concrete made of coal ash also shows the same tendency.

Placing temperature of concrete and time of electrifying

The higher the temperature of concrete when placed, the greater is the initial current which will flow into it. However, the setting time is shortened and thus the total quantity of electric power consumed is less. If concrete is electrified after setting, power consumption is low.

COST OF ELECTRIC CURING

Electric power consumed in electric curing is about 30 to 40 KWH per cu m (23 to 30 KWH per cu yd) of concrete in whole section electrifying and 7 to 10 KWH per sq m (0.65 to 0.90 KWH per sq ft) of form area of concrete for partial electrifying. The effect of electrode spacing on power consumption is shown in Fig. 11.

Execution cost of electric curing is 7 to 10 percent of construction cost for plain concrete, and 10 to 15 percent of construction cost for reinforced concrete, including 20 percent capital cost for equipment. In general, cost is affected by size, form, layout of structures and climatic conditions, and is estimated to be about 30 to 40 percent of that for steam curing or heated enclosures including construction cost of temporary covering. Components of cost for electric curing are given in Table 1.

TABLE 1—COST ALLOCATION FOR ELECTRIC HEATING OF CONCRETE

Type of concrete	Structure	Concrete, cu m	Materials, percent	Labor, percent	Electric power, percent	Equipment depreciation, percent
Plain	Bridge abutment	50	5.4	78.2	15.9	5.9
Reinforced	Coal bin	297	15.3	63.1	6.1	15.5
	Coal bin	204	12.9	53.1	2.7	31.3

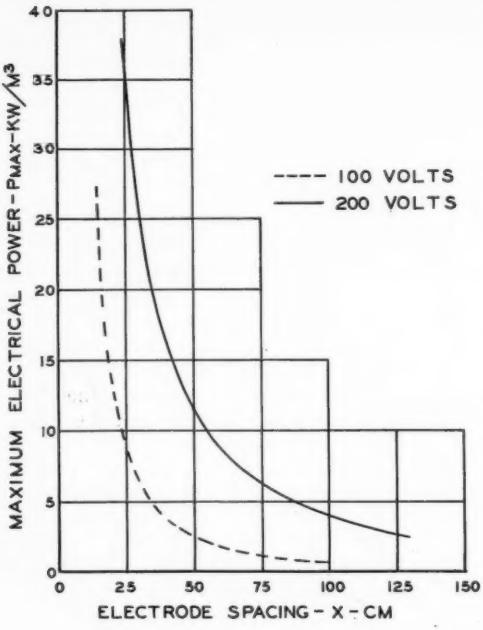


Fig. 11—Relation between electrode spacing and maximum power consumption

ECONOMY IN ELECTRICAL REQUIREMENTS

The following measures are suggested to keep electric power consumption at a minimum.

1. Minimize mixing water and make dry mix concrete.

2. Begin electrifying after concrete has been left stand 2 or 3 hours, or after surface water has left the surface of the concrete, especially in case of a wet mix as in heavily reinforced concrete. If there is plenty of time for curing electrify after setting.

3. Minimize the range of rise of concrete temperature by pre-warming the concrete before or while mixing.

4. Keep the voltage comparatively low.

5. Operate switches often, at 30- to 40-minute intervals, especially in partial electrifying.

6. Even in whole section electrifying, the temperature rise should be small, and continuous electrifying must not be done by frequent reduction in current.

7. Use simple means to prevent cooling if initial concrete temperature is high. Let concrete alone for several hours and electrify after complete setting.

8. If transformer capacity is insufficient, electrify partially by dividing circuits and switches when electrodes and branch wires are being placed. Thus, electrical power requirements can be minimized.

CONCLUSIONS

To get the best effect with minimum power in electric curing of concrete, it is necessary to use the least possible quantity of mixing water and well-graded fine and coarse aggregates, minimize admixtures, and raise the temperature of the concrete mixture in cold weather.

In electric curing, operations should be designed and performed to secure uniform temperature rise in every portion of the concrete. In addition to the general conditions stated above, it is necessary (1) to space similiar electrodes closely, generally not more than $y = 15$ to 20 cm (6 to 8 in.), and to space out electrodes of dissimilar nature to let the temperature rise more slowly, (2) operate switches frequently, turning the current on and off, (3) place concrete in such a way as to raise temperature uniformly and to minimize the necessity of raising the temperature by use of electric current (in cold weather concreting, mixing concrete with heated aggregates and hot water, it is difficult to make concrete temperature uniform in every batch; therefore, it is more desirable to raise concrete temperature uniformly even though somewhat lower in absolute heat degrees), and (4) establish proper length of electrodes and main lead wires.

When electrodes are set vertically or horizontally beyond a certain limited length, even within the safe current value of the electrode, if the feeder ends are all at one side then the voltage falls at the opposite ends and the current becomes weaker and consequently the rise in temperature is delayed. In such case, current should be fed to the opposite ends of every electrode reciprocally, or the electrodes must be shortened and fed directly.

The same considerations apply to main wiring also. If the main lead wire is long and has many branch wires attached to it, voltage drop becomes great and rise of concrete temperature is delayed at the end of the main wire. In such cases, main wiring must be arranged in a loop circuit or the feed must be secured by several parallel circuits from transformer to switches.

SUMMARY

To promote the development of the rich natural resources of Hokkaido, Japan, winter concreting is considered necessary. For river structures it is particularly desirable to do the work in winter when river stages are low. With ordinary curing, the rigorous climate greatly increases the expense and creates difficulties in regard to fuels. However, the electric curing method has proved to be both profitable and useful.

Greater efficiency would result through the use of automatic switches actuated by changes in temperature in the concrete. Large capacity auto-transformers whose voltage is variable from 300 to 50 volts would permit greater flexibility in the electrical heating system.

Field Problems in Constructing a Prestressed Concrete Bridge*

By CLAIR L. JOHNSON†

SYNOPSIS

Discusses design considerations for a prestressed concrete bridge in which the girders were assembled from precast blocks grouted after completion of the members. Field experience showed that a reduction in the number of shapes of blocks used was desirable and resulted in lower over-all cost. Breakage of blocks and difficulties in grouting between blocks were overcome by modification of assembling technique, changes in design and the development of re-usable joint covers. Bid prices on three successive jobs indicate that with familiarity with prestressed construction the cost tends to drop.

INTRODUCTION

The Earl L. Clark Memorial Bridge is on John R Road, Royal Oak Township, just north of Detroit, Michigan. It was built by L. A. Davidson of Lansing, for the Red Run Drainage Board, incidental to a flood control project of the Corps of Engineers, Dept. of the Army. It is 130 ft 4 in. long, divided into 3 approximately equal simple spans. The crown elevation is about 25 ft above the 40-ft wide channel bottom.

The structural elements of the superstructure are job assembled I-section prestressed concrete girders post-tensioned with galvanized bridge strands. The wearing surface was placed on the side-by-side girders. Then transverse prestress was applied which caused the superstructure to become, in effect, a hollow-cored slab. (Fig. 1)

DESIGN CONSIDERATIONS

The girders of I-shaped blocks were designed to be assembled on the job, and jointed by pouring a rich grout into spaces between the blocks (Fig. 2)

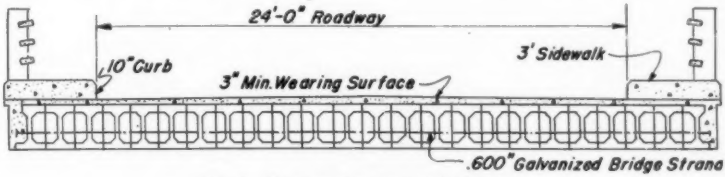


Fig. 1—Cross section through bridge

*Presented at the ACI 48th annual convention, Cincinnati, Ohio, February 26, 1952. Title No. 48-49 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952, *Proceedings* V. 48. Separate prints are available at 35 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Partner, Johnson and Anderson, Engineers, Pontiac, Mich.

Fig. 2—Blocks jointed by pouring rich grout into joints

created by casting a depression in the face of the block (Fig. 3). All blocks were machine made except the tapered end block and the depressor blocks. The latter were to be machine made but the plant men decided that the numbers required would not warrant re-setting the machine.

DESIGN ERRORS

Too many special blocks were specified, requiring many runs of a few blocks each, an undesirable condition from the standpoint of the manufacturer. Present design specifies only three block types; all machine-made. This should reduce future bid prices on the block.

Tolerances expected in the finished block were unreasonable and although the block were uniform, it was practically impossible to assemble a girder of sufficient uniformity to allow placing as planned. Therefore the girders in place occupied slightly more space transversely than anticipated.

CONSTRUCTION CONSIDERATIONS

Block breakage was excessive in the first few days. Some was due to improper loading which was quickly and effectively corrected. Other breakage resulted from attempts to align the girder before prestressing, that is, overtensioning the strands prior to grouting, and trying to force the girder into perfect alignment. After the limitations of the blocks were established and a new tolerance determined, no further trouble was experienced.

Pouring the joints in the first girder was a problem since it was necessary first to plaster the outside of the block. This slowed down assembly, but assured full bearing on the joint. Improved design has reduced this problem considerably. Covering the joints, when necessary, with salvageable material further reduces the time and cost of assembly. The girder must be assembled on a level surface, otherwise the completed member will have an excessive permanent camber.

Fig. 3—Depression cast in face of block to form opening for grout

The first girder assembled was loaded with the equivalent of dead load plus 1½ times live load and impact (Fig. 4). The deflection was

¾ in. which increased to 1 in. in 15 days and then remained unchanged. No cracking occurred.

The strands were tensioned in three stages. The girder was assembled, the strands were placed and then the nuts were tightened with a hand wrench just enough to assure a snug fit between blocks. Approximately 5000 lb per girder was applied after joints were grouted. This prestress fell off almost immediately. Final tensioning by means of hydraulic rams was done after the grout had attained its set; a minimum of three days.

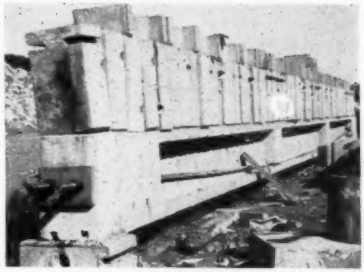


Fig. 4—Girder loaded with equivalent of dead load plus 1½ times live load and impact

After final prestressing, highest stress is in the bottom fiber. As camber is produced by the tensioning, the dead load of the girder becomes effective, tending to offset the fiber stresses due to prestressing. This serves as a test of each girder.

Placing the girders presented no problem. The contractor used an 18 in. I-beam as a spreader, with loops of wire rope for hookers.

COST CONSIDERATIONS

On the basis of the few bids received in this area, the cost of prestressed concrete is apparently dropping. The first bid was on an 80-ft single-span bridge, whose estimated cost was $52,000. Alternate bids were taken for steel and for prestressed concrete. The low bid for steel was $50,000 and for prestressed concrete was $57,500. This represents an increase of about 11 percent over estimate and 15 percent over the steel price.

On the second bridge, advertised for prestressed concrete only, estimated cost $78,000, the low bid was $99,000, or 27 percent high. On the third bridge, estimated cost $154,000 based on the unit prices for the second job, the low bid was $136,000, or 12 percent low. These figures are of little importance, as they were not received at the same time, but do seem to indicate that better bids may be expected in the future.

Some interesting figures on assembly costs were given by the contractor. Labor for assembling and prestressing the second span was 12 man hours per ton of steel which compares very favorably with the 152 man hours per ton of steel reported on an earlier job using individual wires. Table 1 shows a breakdown of the cost of one girder.

Not included are cost of placing, overhead, supervision or profit. The bid price was approximately $430.00.

SUMMARY

To overcome difficulties encountered, different standard block sections are now incorporated in the design. The top flange is narrower than the solid

TABLE 1—APPROXIMATE COST OF MATERIALS AND LABOR PER GIRDER

em	Number	Unit cost	Cost per item
Block	63	0.90	$56.70
Strands	2	40.00	80.00
Bearing plates	2	16.00	32.00
Grout			2.00
Labor, man hours	12	2.60	31.40
		Total	$202.10

end and bend up blocks and when assembled, leaves a 2 in. longitudinal opening between the girders except at the ends and at the bend up points. This opening is formed and filled to the bottom of the top flange when the deck is placed (Fig. 5). Transverse prestressing strands are placed in openings provided in the top of the block and enclosed in concrete when the deck is placed. This permits greater tolerance in assembly, eliminates transverse prestressing problems even in skewed bridges and incorporates the wearing surface into a composite section for the completed structure.

This composite effect was not considered in the John R Bridge, but it unquestionably exists and adds rigidity to the structure. Allowable deflection is 1 part in 800 under full load plus impact. The best loading which could be arranged to approximate design load gave a deflection of approximately 1 part in 8000.

REFERENCES

1. Johnson, C. L., "The John R. Bridge," *Proceedings*, U. S. Conference on Prestressed Concrete, M. I. T., Aug. 15, 1951.

2. Johnson, C. L., "Prestressed Girders of Machine-Made Block," *Concrete*, Dec. 1951.

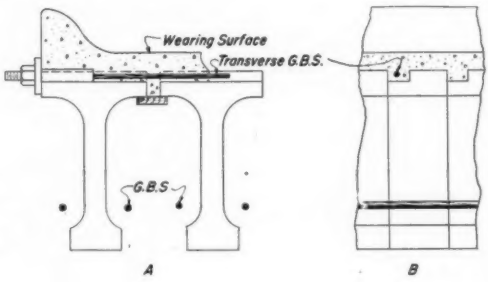


Fig. 5—New block design showing method of integrating slab with blocks

Title No. 48-50

Economy and Concrete Beams*

By HERBERT A. SAWYER, JR.†

SYNOPSIS

Relationships and charts are presented from which may be readily determined the cross-sectional dimensions for maximum economy of materials for a beam or slab with given span and loading. Whitney's theory is used to express beam strength. The steel saving possibilities of economical dimensioning are considered and found especially important.

These optimum dimensions almost never correspond to balanced design, but depend on relative material costs, relative material strengths, relative length of beam reinforced for diagonal tension, and relative importance of beam dead weight.

Optimum dimensions for the Whitney theory also serve as a general guide to efficient design by the elastic theory.

INTRODUCTION

What cross-sectional dimensions make the most economical beam for a given bending moment? For beams of a single material the answer is almost obvious—place the material at the largest moment arm from beam center which will not impair stability and so that as much as possible of it reaches the limit of its useful strength simultaneously. Many writings on reinforced concrete have passed off the subject of economy with a similar quick generalization—that balanced design is the most economical. Yet this principle has never been proved for a beam made up of two materials of different relative costs and strengths, and we all know it is not true for T-beams. Several investigators, including Charles S. Whitney,[1] N. J. Cochrane,[2] and the late Dean Peabody,[3] have demonstrated its untruth in more or less special cases. Homer Hadley[4] has recently stated that this principle is generally untrue for present costs, which is borne out by the following investigation.

The question then arises—what general principles must be followed to achieve maximum economy as well as the required strength in the design of a reinforced beam? An answer to this question is especially important in these times of material shortages and changing relative costs. This paper attempts to supply that answer.

OPTIMUM STEEL PERCENTAGE FOR ECONOMY

There are many different opinions as to how to simply and accurately

*Presented at the ACI 48th annual convention, Cincinnati, Ohio, Feb. 27, 1952. Title No. 48-50 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952. *Proceedings* V. 48. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Sept. 1, 1952. Address 18263 W. McNichols Rd., Detroit 19, Mich.

†Member American Concrete Institute, Associate Professor of Civil Engineering, University of Connecticut, Storrs.

express the strength of concrete beams. For the purposes of this investigation, Whitney's[5,1] expressions for beam strengths are assumed to be true within the ranges of strengths of steel and concrete used in practice. As will be seen, the dimensions derived on this basis will throw light on the proportions to be used for economy with the common elastic theory.

This paper considers the economy of beams or slabs carrying given live loads over given spans, considering only the cost of the structural skeleton. Any change in cost of the other parts of the structure with change in beam dimensions is neglected; of course such changes should be considered in any practical case. Variation in cost of forms from variation of beam dimensions is usually almost negligible, and hence will be neglected. I. E. Morris[6] has already described the situation where variation in form costs is not negligible —when beam depths are so shallow that beam forms are eliminated.

Inge Lyse[7] and R. L. Bertin[8] have discussed the saving of steel by modification of its allowable stress.

According to Whitney's expressions, addition of tensile steel to a balanced beam will not increase its strength, and other expressions for ultimate strength show but very slight increase in strength for such additional steel. Therefore such additional steel is uneconomical, and consideration will be limited herein to beams which have less steel than that required for balance. The term "balanced design" means, for this paper, proportions resulting in theoretical simultaneous flexural failure of steel and concrete at a cross section.

Fig. 1 shows a typical under-reinforced beam with the stress pattern and notation used by Whitney[1] to express its ultimate strength. For given moment and materials, three variables are involved in proportioning this cross section: the breadth b, the depth d, and the ratio of steel area to concrete area p, any one of which is dependent on the other two.

The most economical beam in theory would have infinite d and infinitesimal values of b and p—an entirely impractical solution. If d is held constant, the most economical b and p generally indicate slightly less steel than needed

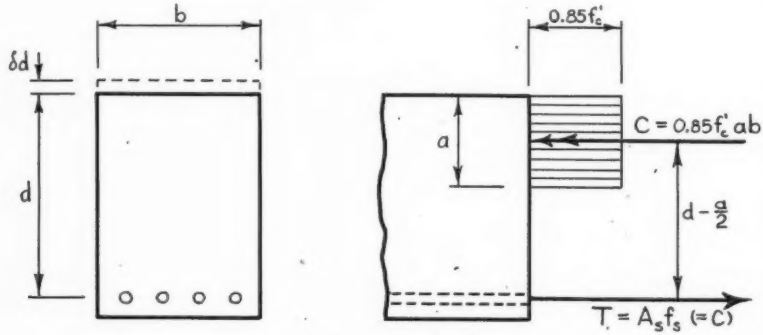


Fig. 1—Whitney stress distribution

for balanced design. However, this value of b almost never allows room for the reinforcement, so this solution has little practical significance. For economical design, the somewhat-fixed least value of b which allows placement of the reinforcement should be used. The great problem, then, is the determination of the most economical values of the depth d or the steel ratio p for fixed breadth.

If, for given moment and types of materials, the costs of the materials of a cross section are plotted against values of p, a general type of curve will be obtained of which Fig. 3 is a typical example. It is clear from examining this curve that, at the point of minimum total cost, the increase in concrete cost for a small decrease in p is equal to the decrease in steel cost, or

$$\delta A_c = -\delta A_s R$$

Also, since the resisting moment is constant

$$\delta M_c = -\delta M_s$$

where subscripts c and s stand for concrete and steel, respectively, A is area, M is moment, and R is the ratio of the cost of adding a unit volume of steel to the cost of adding the same volume of concrete to the structure. Note that this definition of R excludes all placement costs which are constant after placement of the first few yards of concrete or pounds of steel, that is, costs which are not affected by placement of additional concrete or steel.

In this paper these general equations have been solved for the two special cases which are important.

Effect of stirrups included, dead load neglected

This case covers all beams with breadths somewhat fixed by the spacing requirements of the reinforcement, with possible diagonal tensile steel, and with relatively small dead load moments from their own weights. This case is probably the most general in application; it applies to all average-to-short-span rectangular beams and T-beams with negative moment. It even applies closely enough to T-beams with positive moment.

If the beam has reinforcement for diagonal tension, the increase in concrete area in the general equation increases the concrete's resistance to diagonal tension, allowing a decrease in diagonal tensile steel. (See Appendix A.) This saving in cost should of course be included in the general equation. The resulting relationship in its most convenient form is

$$\frac{1}{p} = \frac{1}{\frac{1}{R} - \frac{L_s}{8.5\ mL}} + m \qquad \text{(See Appendix B)}$$

Where

$p = A_s/bd$
L_s = Length of beam requiring diagonal tensile steel
L = Total length of beam
$m = f_s/0.85\ f_c'$ where f_s = yield point of steel
f_c' = cylinder strength of concrete

Values of p given by this relationship may be found from the proper curves of Fig. 2, which have been plotted for this relationship for a value of m of 15.

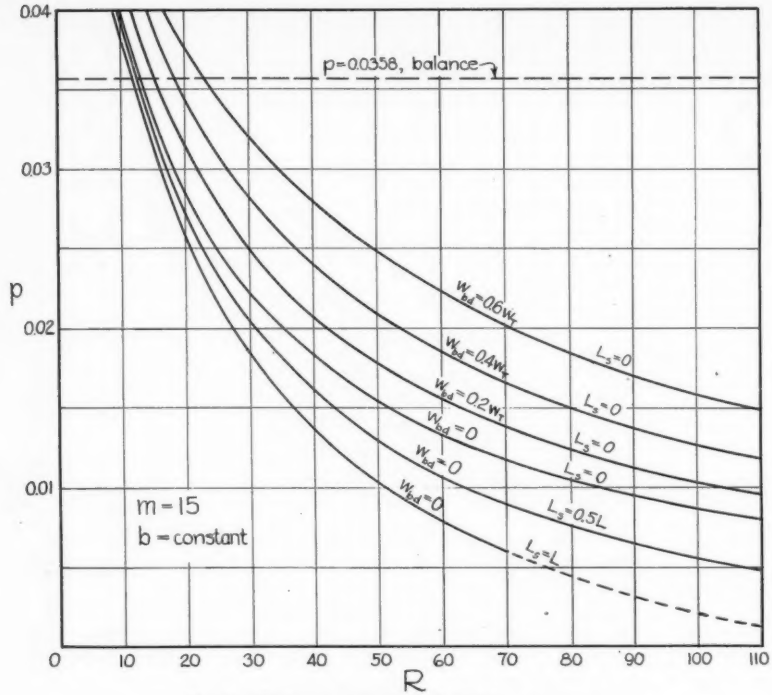


Fig. 2—Optimum proportion of steel for economy

How important is it that the values of p given by this relationship be maintained? In Fig. 3 the results of many designs for a constant moment and breadth and variable p are plotted for no diagonal tensile reinforcement and a cost ratio R of 50, and in Fig. 4 similar results are plotted for half-length diagonal tensile reinforcement and a cost ratio of 80. As can be seen from both curves, p may have any value in the range of from 2/3 to 3/2 of its optimum value with no more than two or three percent added cost for the conditions plotted. In practice, the cost of added structural height to maintain clearance requirements using the low values of p would probably dictate use of values near or over the top of this range.*

The relative cost of the steel and the concrete is also shown for these designs. As would be expected, variation of p within even the above restricted range causes great variation in the relative costs or amounts of steel and concrete. If either material is in short supply, choice of the proper p will do much to help—and without significantly affecting the over-all cost.

*The relative values of d have been plotted in Fig. 3 and 4 to help evaluate this factor. It can be seen that the last few inches of clearance gained in approaching balanced design for a beam of usual depth would add roughly 10 to 20 percent to material costs for the beam. It can be concluded that balanced design should probably never be used, even when clearance is relatively important, except possibly near columns, where negative moment is high.

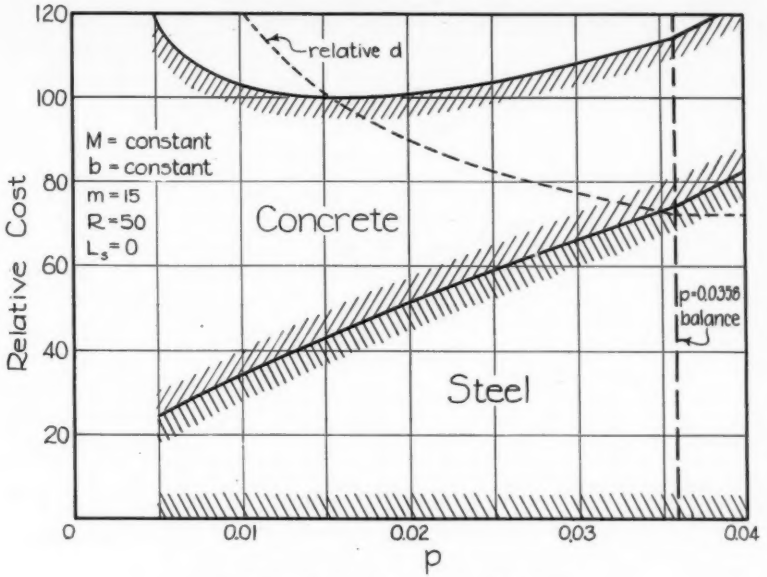


Fig. 3—Cost vs steel ratio, no web reinforcement

No stirrups, dead load effect included

This case covers slabs and long-span beams, which usually have little if any diagonal tensile reinforcement. Also, the dead load of slabs and long-span beams is usually a significant portion of the total load.

If the dead load is important, the increase in resisting moment from adding concrete area is partly absorbed or lost by the increase in bending moment from increase in dead load. If the increase in moment from adding concrete is corrected for this loss, the general equations result in the following relationship

$$R = \frac{\frac{1}{p} - m}{1 - \frac{w_{bd}}{w_T}\left(1 - \frac{pm}{2}\right)} \quad \text{(See Appendix C)}$$

Where

w_{bd} = weight of concrete area bd per unit length
w_T = total load per unit length

If dead-load and live-load factors of safety differ, these loads per unit length should be multiplied by their respective factors of safety.

Values of p given by this relationship may be found from the proper curves of Fig. 2, which have been plotted using a value of m equal to 15.

As in the first case, values of p ranging from about 2/3 to 3/2 of its optimum value will not add significantly to the cost of materials. Clearance is

probably less important than in the first case, but the relative proportions of steel and concrete will obviously vary about as widely within this range. Therefore, if steel is scarce, use of the lower values of p for design will result in great steel savings with almost no change in over-all material costs.

APPLICATION TO ACTUAL BEAM DESIGN

Moment varies extremely throughout the length of all beams, so the question arises—for which value of the moment will the economical dimensions, determined as above, be the most economical for the entire beam?

The solution to this problem comes from the consideration that (for beams of constant concrete area throughout) the optimum value of p given above is the optimum ratio of the total volume of flexural steel in the beam to the volume of concrete in the beam as given by the expression bdL. Therefore the beam may be proportioned for economy by using a value of the moment which is the average moment of resistance of the total flexural steel, assumed to be evenly distributed as tensile steel throughout the beam. Note that this volume should include *all* flexural steel, positive and negative, tensile and compressive, anchorage steel, and steel in all directions for two-way or flat slabs.*

Approximate values of these dimensioning moments may be obtained on this basis for any general type of design. Very approximate values are good enough because of the wide range of economical values of p. The dimensioning moment for a simple beam is approximately $1/10\ w_T L^2$, for a continuous beam $1/12\ w_T L^2$, and for the slab of flat slab (with drop-panel) construction $1/11\ w_T L^2$.

The value of the relationships given in this paper is enhanced by the use of the universal design analysis chart of Fig. 5. All beam dimensions may be directly obtained from such a chart for any combination of moment, materials, and percentage of steel. For actual use the chart should be drawn to larger scale and in more detail.

Fig. 2 shows optimum values of p for an m of 15 only. Dotted lines representing the optimum values of p for various values of m have been plotted on the NW quadrant of Fig. 5, using values of $L_s = 0$ and $w_{bd} = 0$. It is evident that p is almost independent of m for the normal values of m between 10 and 20, and, taking into account the wide range of economical p, it may be shown that p is practically independent of m for all normal values of L_s and w_{bd}. Therefore Fig. 2 is sufficiently accurate for all practical cases.

The ideal procedure for the most economical flexural design by the Whitney theory is as follows:

1. Determine optimum p from Fig. 2. Establish the economical 2/3 to 3/2 range of p on Fig. 5.
2. Calculate $M/0.85\ f_c'$, where M is the dimensioning moment times the factor of safety.

*The one exception is the flexural steel required to maintain the minimum p specified in many codes. For an increase in a beam's concrete area, this type of clause has the effect of increasing the minimum steel area, while, as shown above, the maximum steel area decreases. It would seem that for economy and other reasons these clauses should be revised to make the minimum steel area a specified percentage of the maximum steel area.

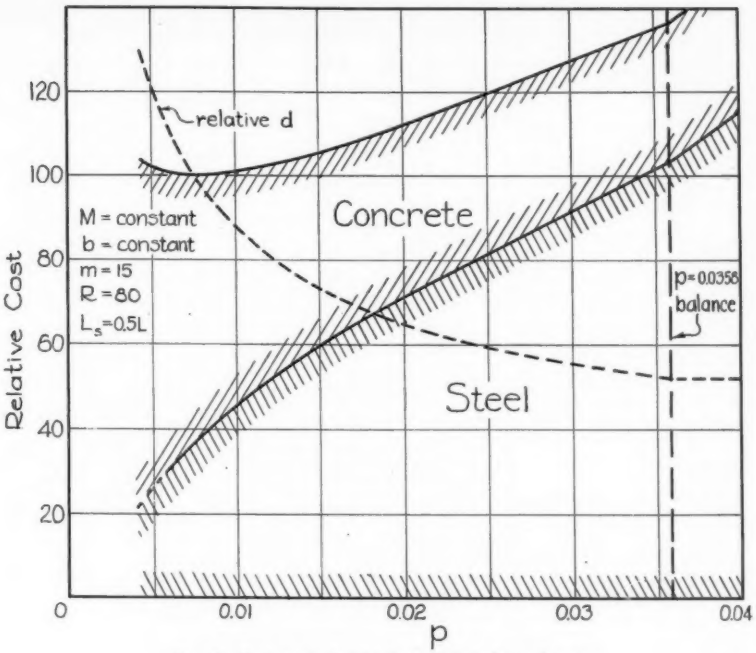


Fig. 4—Cost *vs* steel ratio, including web steel cost

3. (Omit for slabs.) Estimate the probable minimum b into which reinforcement will fit.

4. Entering Fig. 5 at the p axis, work around chart clockwise. Use radial SE quadrant lines for b. Read value of d on lower vertical axis, selecting lower value in the range if clearance is important, and a higher value if steel is to be conserved.

5. Select steel for the ultimate design moment at any section by entering chart at bd^2 just established. Work both clockwise, using radial SE quadrant lines for d, and counterclockwise, using the ultimate design moment for the section, and determine required A_s. For T-beams with compression in flange, use same general procedure—use d selected in step 4, but b equal to effective width of flange, establishing new bd^2. Observe value of pm in NW quadrant; if pm is less than the ratio of flange thickness to d (and it usually is), chart is applicable.

Of course many factors such as architectural requirements or requirements for other stresses may modify this procedure in an actual case.

ECONOMY FOR THE ELASTIC THEORY

Horizontal dotted lines on the NW quadrant of Fig. 5 show values of the moment arm between the resultant tensile and compressive forces in the beam, relative to the depth. For balanced design the Whitney theory allows more steel and specifies a shorter arm ($j = 0.732$) than the elastic theory. But as the steel percentage is lowered toward its economical values, the

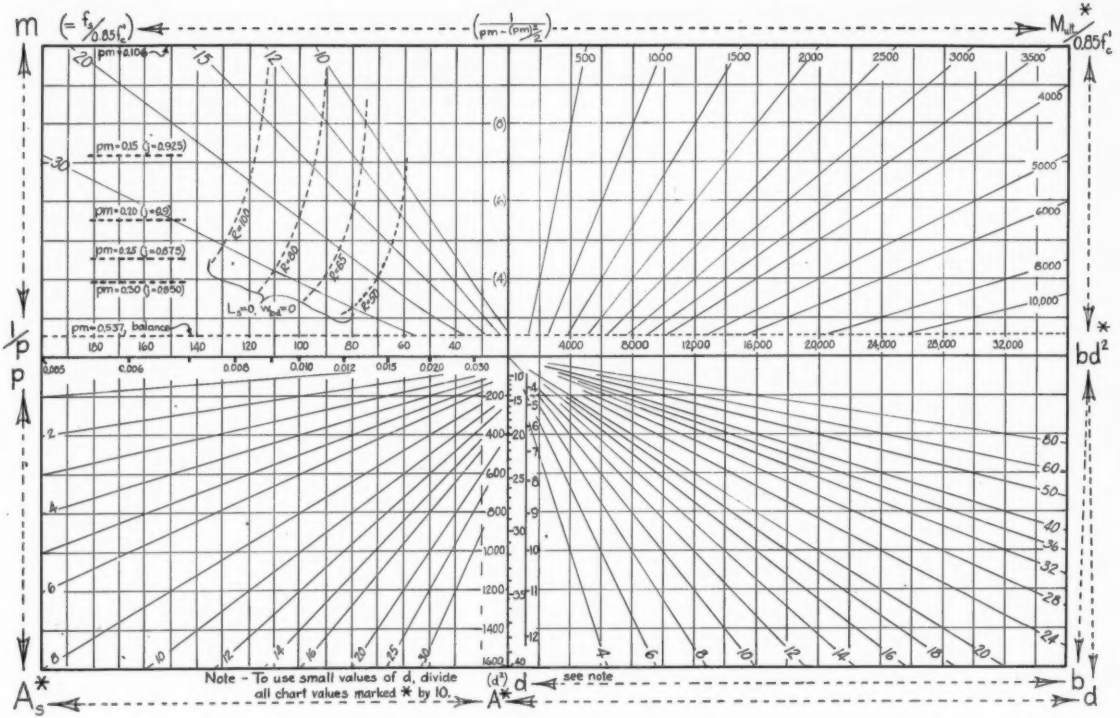


Fig. 5—Flexural design chart, Whitney theory

moment arm increases, and in the most economical ranges the moment arm given by the Whitney theory, Fig. 5, generally exceeds the moment arm given by the elastic theory. Thus, for *economical* steel percentages the Whitney theory credits the steel with more effectiveness than the elastic theory. It follows that the Whitney theory tends to use less steel than the elastic theory for economical design with b fixed.

This conclusion is also demonstrated by the calculation of the most economical p for the elastic theory. Peabody[3] did this calculation, neglecting influence of diagonal tensile steel and dead weight, and found optimum values of p slightly higher than those found herein for the Whitney theory, but still within the economical 2/3 to 3/2 range.

By coincidence, balanced elastic theory design gives steel percentages much nearer the economical range of p than the percentages for balanced Whitney theory design. But obviously these percentages for balanced elastic design, depending as they do only on the physical properties of the materials, are completely worthless as a guide to economical percentages, primarily depending as they do on relative material costs, relative length reinforced in diagonal tension, and relative importance of dead weight. The values of p of Fig. 2 may be used as a general guide to economical design by the elastic theory, provided these values do not exceed the values for balanced elastic design.

The *ratio* of steel cost to concrete cost has risen roughly 25 percent in the past five years. A continuation of this trend will make balanced elastic design generally uneconomical.

CONCRETE STRENGTH AND ECONOMY

For economical design the concrete stress does not govern, so the possibility arises that further economy may be gained by lowering the strength of the concrete. According to Whitney[1]

$$M = pf_s bd^2 \left(1 - \frac{pm}{2}\right)$$

or, if only the concrete strength in a given beam is changed:

$$M = K\left(1 - \frac{pm}{2}\right)$$

where K is constant. If p has a value of 0.0125, which is normal for economical design, and m is increased from 10 to 20, indicating a 50 percent reduction in concrete strength, the above relationship shows that the flexural strength of the beam is reduced only about 6 percent. The savings in cement and inspection costs which would result from a realization of this unimportance of concrete strength could possibly outweigh any savings from economic dimensioning many times. However, concrete strength is important for bond, diagonal tension, durability, and for resisting the high negative moments localized at columns, so no hasty lowering of standards is warranted.

CONCLUSIONS

1. Balanced design by the Whitney theory is practically never economical of materials, and balanced design by the elastic theory is often not economical.

2. The percentage of steel for economical design depends primarily on the ratio of the costs of steel and concrete, the amount of diagonal tensile reinforcement, and the relative importance of the beam's dead weight, and averages less than half the percentage for balanced design for the Whitney theory.

3. This steel percentage may vary from 2/3 to 3/2 of its optimum value in a given case without appreciable adverse effect on economy of materials. Use of the lower limit decreases steel quantities, and use of the higher limit tends to reduce the height of the structure.

4. In the range of economic steel percentages, the Whitney and elastic theories are nearly equivalent, the Whitney theory specifying a slightly greater moment arm or j.

5. Economical design for the Whitney theory is facilitated by use of a universal design chart.

6. For economic design, the flexural strength is almost independent of concrete strength, therefore for flexural design the concrete may be regarded as an inexpensive but heavy filler material to provide moment arm for the steel.

REFERENCES

1. Whitney, C. S., "Plastic Theory of Reinforced Concrete Design," *Transactions*, ASCE, V. 107, 1942, p. 259 and 325.

2. Cochrane, N. J., "Economics of Reinforced Concrete Sections," *Journal*, Institution of Civil Engineers, V. 28, No. 3, Jan. 1945, pp. 155-159.

3. Peabody, Dean, *Reinforced Concrete Structures*, Second Edition, John Wiley and Sons, New York, 1946, pp. 109-115.

4. Hadley, Homer, "Skepticism Regarding the Plastic Theory of Design," ACI JOURNAL, Sept. 1951, *Proc.* V. 48, p. 102.

5. Whitney, C. S., "Design of Reinforced Concrete Members Under Flexure or Combined Flexure and Direct Compression," ACI JOURNAL, Mar.-Apr. 1937, *Proc.* V. 33, pp. 483-498.

6. Morris, I. E., "Economy in Structural Design," ACI JOURNAL, Jan. 1948, *Proc.* V. 44, pp. 349-360.

7. Lyse, Inge, "Relations Between Quality and Economy of Concrete," ACI JOURNAL, Mar.-Apr. 1933, *Proc.* V. 29, pp. 325-343.

8. Bertin, R. L., "Saving Steel in Reinforced Concrete Design," ACI JOURNAL, Feb. 1942, *Proc.* V. 38, pp. 281-288.

APPENDIX A—DIAGONAL TENSION*

$A_v = \dfrac{Vs'}{f_v jd}$ from ACI Building Code

$$\text{(Vol.)} = \frac{A_v}{s}\ \text{(length of stirrup)} = \frac{V'}{f_v jd}\left(d + \frac{b}{2}\right)$$

Where (Vol.) = Volume of diagonal tensile steel per unit length of beam

Assume that $\dfrac{b}{2} = \dfrac{d}{4}$ Then $\text{(Vol.)} = \dfrac{V'}{0.8jf_v}$

But $V' = V - 0.03f_c' jbd$

For ultimate load design, substitute ultimate strengths for working stresses:

$0.08f_c'$ for $0.03f_c'$ (assuming factor of safety = 2.67)

f_s for f_v where f_s = yield point of steel

$$\text{Then (Vol.)} = \frac{V - 0.08f_c' jbd}{0.8jf_s}$$

$$\text{Differentiating, } \delta\text{(Vol.)} = \frac{-f_c' b}{10f_s}\,\delta d$$

APPENDIX B—STIRRUPS INCLUDED, DEAD LOAD NEGLECTED

1. Add δd to top of beam (Fig. 1), then addition to resisting moment is $\delta M_c = \delta d A_s f_s$ since concrete stress block moves up δd.

Increase in cost per unit length along beam

$= \delta d\ b\ C_c -$ (saving in diagonal tension steel)

$$= \delta d\ b\ C_c - \frac{L_s f_c' b}{L\ 10 f_s}\,\delta d\ C_s \quad \text{(From Appendix A)}$$

Where

C_c = Cost of concrete per unit volume added

C_s = Cost of steel per unit volume added

2. Remove δA_s from flexural steel (no change in d), then loss in resisting moment is $\delta M_s = \delta A_s f_s (d - a)$

$$\text{Decrease in cost per unit length along beam} = \delta A_s C_s = \frac{\delta M_s}{f_s (d - a)} C_s$$

3. For minimum cost $\delta M_c = \delta M_s$, and increase in cost equals decrease in cost

$$\frac{\delta M_s}{f_s (d - a)} C_s = \frac{\delta d\ A_s f_s}{f_s (d - a)} C_s = \delta d\ b\ C_c - \frac{L_s f_c' b}{L\ 10 f_s}\,\delta d\ C_s$$

Or

$$\frac{A_s}{b\ (d - a)} = \frac{C_c}{C_s} - \frac{L_s}{L}\frac{f_c'}{10 f_s}$$

$$\text{Since } m = \frac{f_s}{0.85\, f_c'};\quad p = \frac{A_s}{bd};\quad R = \frac{C_s}{C_c}:\quad \frac{p}{1 - pm} = \frac{1}{R} - \frac{L_s}{8.5m\ L}$$

Or

$$\frac{1}{p} = \frac{1}{\dfrac{1}{R} - \dfrac{L_s}{8.5m\ L}} + m$$

*ACI Building Code notation unless otherwise noted

APPENDIX C—STIRRUPS NEGLECTED, DEAD LOAD INCLUDED

1. Add δd to top of beam (Fig. 1).

   Moment lost from added dead load $= \frac{\delta d}{d}\frac{w_{bd}}{w_T} M = \frac{\delta d}{d}\frac{w_{bd}}{w_T} f_s A_s \left(d - \frac{a}{2}\right)$

   Net increase in resisting moment $= \delta M_c = \delta d\, A_s f_s - \frac{\delta d}{d}\frac{w_{bd}}{w_T} f_s A_s \left(d - \frac{a}{2}\right)$

   Increase in cost per unit length $= \delta d\, b\, C_c$

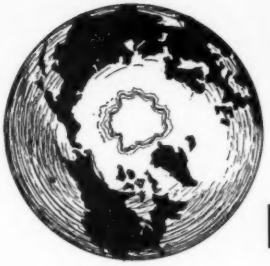
2. Same as item 2, Appendix B.

3. For minimum cost $\delta M_c = \delta M_s$, and increase in cost equals decrease in cost

$$\frac{\delta M}{f_s (d - a)} C_s = \left[\delta d\, A_s f_s - \frac{\delta d}{d}\frac{w_{bd}}{w_T} f_s A_s \left(d - \frac{a}{2}\right)\right] \frac{C_s}{f_s (d - a)} = \delta d\, b\, C_c$$

Or $$\left[1 - \frac{w_{bd}}{w_T}\left(1 - \frac{a}{2d}\right)\right] \frac{A_s C_s}{b (d - a)} = C_s$$

Or $$R = \frac{\frac{1}{p} - m}{1 - \frac{w_{bd}}{w_T}\left(1 - \frac{pm}{2}\right)}$$



Letters from Readers*

BY WAY OF SYNOPSIS

FREDERICK L. BROWNE discusses a gravimetric method for computing air in concrete.

Fire resistance of prestressed concrete is considered by RALPH E. BRADDON and A. E. CUMMINGS.

A method for determining frame constants is given by WARREN E. MCNELY and LARGY V. RUDCHENKO.

Computation of Air in Air-entraining Concrete by the Gravimetric Method (LR 48-26)

When it is desired to determine by the gravimetric method the amount of air in a batch of air-entraining concrete it is generally considered necessary to know the theoretical unit weight of the concrete on an air-free basis. A simple formula is proposed by which the air content may be determined without knowing the theoretical unit weight on an air-free basis.

$$P_2 = 100 - \frac{W_2}{W_1}(100 - P_1) \quad \ldots\ldots (1)$$

Where

W_1 = the unit weight of concrete containing the design percentage of air
W_2 = the unit weight of concrete by test
P_1 = the design percentage of air
P_2 = the actual percentage of air determined by test

This formula is derived from ASTM C 138-44:

$$P_1 = \frac{W_0 - W_1}{W_0} \times 100 \quad \ldots\ldots (2)$$

where

W_0 = the theoretical unit weight on an air-free basis.

Similarly

$$P_2 = \frac{W_0 - W_2}{W_0} \times 100 \quad \ldots\ldots (3)$$

Solving for W_0:

$$W_0 = \frac{100\ W_1}{100 - P_1} \quad \ldots\ldots (4)$$

$$W_0 = \frac{100\ W_2}{100 - P_2} \quad \ldots\ldots (5)$$

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952, *Proceedings*, V. 48. Separate prints are available at 35 cents each. Address 18263 W. McNichols Rd., Detroit 19, Mich.

Then

$$\frac{W_1}{100 - P_1} = \frac{W_2}{100 - P_2}$$

Solving for P_2 we have

$$P_2 = 100 - \frac{W_2}{W_1}(100 - P_1) \quad \ldots\ldots\ldots (1)$$

Take the example of the following mix designed to contain 4.5 percent entrained air.

	Solid volume, cu ft	Weight, lb
Air	1.215	0
Water	4.544	284
Cement	2.528	497
Aggregate	18.713	3019
Total	27.000	3800

The unit weight for concrete containing 4.5 percent air is

$$W_1 = \frac{3800}{27} = 140.7 \text{ lb per cu ft}$$

If in a unit weight test on the fresh concrete it weighs 140.7 lb per cu ft we know we have entrained 4.5 percent air.

Should the unit weight by test be 143.0 lb per cu ft substitute this value for W_2 in Eq. 1 and

$$P_2 = 100 - \frac{143.0}{140.7}(100 - 4.5) = 3.0 \text{ percent}$$

The concrete actually contains 3.0 percent air instead of the 4.5 percent air for which the mix was designed.

Eq. 1 is well adapted to use at batch plants where mixes are designed to contain a certain amount of air and the theoretical air-free unit weight is not readily available.

If it is desired to use the theoretical unit weight on an air-free basis in the computation it may be accomplished using Eq. (1) by setting $P_1 = 0$ and $W_1 = W_0$. Thus the theoretical unit weight in the example above is

$$\frac{3800}{27.000 - 1.215} = \frac{3800}{25.785} = 147.4 \text{ lb per cu ft}$$

Should the unit weight by test be 140.7 lb per cu ft then in Eq. 1

$$P_2 = 100 - \frac{140.7}{147.4}(100 - 0) = 4.5 \text{ percent}$$

It is believed that this method of computation is simpler and more universally adaptable to general use than the method of computation prescribed by ASTM C 138-44.

FREDERICK L. BROWNE,
Chief, Concrete Branch,
North Pacific Division Laboratory, Corps of Engineers

Fire Resistance of Prestressed Concrete (LR 48-27)

One aspect of the application of prestressed concrete has not been investigated very seriously, as far as the writer has been able to ascertain.

Consideration of fire code requirements led the writer to ask an experienced metallurgist about the behavior of high-tensile steel wire of the class usually considered practical for prestressing. He explained that the high stress value of the wire is built up, for the most part, by the drawing or stretching both before and during installation. This type of strength increase is highly sensitive to heat, so that subjecting the wires to temperatures of 700 to 1000 F and certainly 1200 F for a very few minutes would cause extensive change in the metal and return it to the strength category of mild steel. Without data it seems not improbable that a temperature of 1000 F could reach the wire with a room temperature of 1550 F, which is designated for a 30-minute exposure. This reclassification of strength could be completely disruptive to the characteristics of a prestressed member. Failure of a member could be complete with only a small spot subjected to critical heat.

From this conference, and a further check

with two steel company men, it seems necessary to urge most earnestly that some comprehensive tests be made before some bad behavior shows up in practice. Bridges and other outdoor structures are certainly good places to use the prestressing method, but in buildings, especially in industrial and process structures possibly subject to fire and heat, its application seems precarious. Most particularly it seems wrong to leave the prestressed wires exposed in a building, as was proposed for a job noted recently. (Galvanizing of the wires was to insure against corrosion.)

Figures have been purposely omitted because fire hazard and stress values are relative matters subject to codes and specifications. Prestressing requires close limits of performance to be effective. Progress in science is best served by attention to all factors which come to light.

RALPH E. BRADDON,
Structural Engineer,
Cleveland, Ohio

While it is true that comparatively little experimental work has been done on fire resistance of prestressed concrete, most European engineers are of the opinion that the fire resistance of prestressed concrete is satisfactory as long as the concrete cover over the high strength steel wire is equivalent to that required by building codes and specifications. For example, the question of protective covering is set forth in Section 507 of the ACI Building Code and the concrete cover required by the code should be sufficient for any ordinary fire hazards.

A year or so ago Dutch engineers subjected a full-sized prestressed concrete beam to high temperature as in a fire. Measurements were made to determine the behavior of the beam under the influence of the heat. The beam was subjected to a temperature of about 1300 F at its bottom surface for 105 minutes but the temperature in the concrete at a depth of about 5 cm above the bottom of the beam reached only 400 to 500 F.

Although engineers on the continent of Europe have not been disturbed about fire resistance of prestressed concrete, the engineers at the British Building Research Station near London have been working on the subject for several years. Recently they started an extensive program of fundamental research in collaboration with the Joint Fire Research Organization of England. Results of the preliminary tests did not indicate anything alarming. The writer has discussed fire resistance with the Freyssinet organization in Paris and they do not feel that any of the test data published to date indicate a serious weakness of prestressed concrete in this respect.

A. E. CUMMINGS, Chairman,
Committee 323, Prestressed Concrete

Of interest to this discussion is a brief article which appeared on p. 100, *Concrete and Constructional Engineering*, London, England, March 1950:

"Some tests were made recently at Maastricht, Belgium, to determine the resistance to fire of concrete beams prestressed with cables. A few previous tests on full-size beams made on the Continent gave satisfactory results within the range of temperatures generally expected when considering the resistance to fire of structures. The beam tested at Maastricht had a span of 38 ft and was prestressed by a Magnel-Blaton cable. During the test the beam carried its working load of about 410 lb per ft. The duration of the fire was 3½ hours and the greatest temperature recorded on the bottom of the lower flange was 1590 F. The greatest additional deflection during the test was about 1/8 in., and the additional permanent deflection after the beam had cooled was negligible."

—EDITOR

Frame Constants for Nonprismatic Members (LR 48-29)

A common problem confronting the designer of reinforced concrete structures is that of the analysis of nonprismatic members. Since most rigid frames are analyzed by the method of moment-distribution, it becomes necessary to find not only moments in members but also stiffness and carry-over factors before starting the actual distribution.

Tables containing the above-mentioned factors are available for most cases of nonprismatic members, those having straight or parabolic haunches. However, there often arises the situation where a member is nonprismatic but has neither the ordinary straight nor parabolic haunches, and other means are required for finding the necessary fixed-end moments, stiffness factors, and carry-over factors. An example of this type of problem might be a beam notched for a trench, or a roof-girder following the slope of the roof.

The most common solution to such a problem is by the method of area-moments, as is illustrated in the Portland Cement Assn. *Handbook of Frame Constants*. Another approach is the column analogy, which the authors believe to be the simplest solution. The usual column analogy solution, however, assumes the load applied to the analogous column to be divided into rectangular segments, thereby approximating the actual curved load. The objection to this part of the method is that the result is often too approximate if the divisions are not sufficiently small, or too involved when a more accurate solution is desired. Instead of this the authors suggest integration of the moment curves which gives not only an exact analysis but also in many cases a simpler solution. To the knowledge of the authors, this particular solution has not been presented in any publication.

The validity of the proposed method of analysis is demonstrated by the solution of a member which may be checked by the tables of haunched-beam constants published by the Portland Cement Assn. Then the case of a girder following the roof-slope is presented in Example 2. It is believed that by following these examples a designer can analyze any nonprismatic member without undue difficulty.

The given examples indicate the repeated use of a number of integrals. Accordingly, these integrals are presented as general formulas with evaluations given for any nonprismatic member having straight haunches. For other types of nonprismatic members, the proper evaluation of these integrals must be found. The formulas following are evaluated for members having unit length, depth and moment of inertia as shown in Fig. 1. Axis *Y-Y* is always taken at the deep end of the haunch.

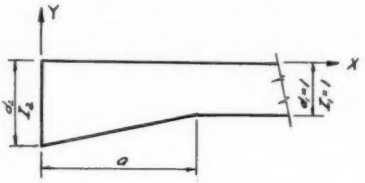


Fig. 1

Total length of beam $L = 1$

$d_1 = 1$ and $I_1 = 1$ at small end of haunch

$$r = \frac{d_2 - d_1}{d_1} = d_2/d_1 - 1 = d_2 - 1$$

$$Ln\,(1 + r) = 2.3026\ \text{Log}\,(1 + r)$$

$$\int_0^a \frac{dx}{I_x} = \frac{a\,(2 + r)}{2\,(1 + r)^2}$$

$$\int_0^a \frac{x^2 dx}{I_x} = -\,(a/r)^2\left[\frac{r\,(2 - r)}{2} - Ln\,(1+r)\right]$$

$$\int_0^a \frac{x dx}{I_x} = \frac{a^2}{2\,(1 + r)}$$

$$\int_0^a \frac{x^3 dx}{I_x} = (a/r)^4 \left[\frac{r\,(r^2-3r-6)}{2} + 3\,(1+r)\,Ln\,(1+r)\right]$$

Using the above formulas, a nonsymmetrical, nonprismatic beam is solved for the beam constants as follows:

EXAMPLE 1

$r = d_1 - 1 = 1$; $a/r = 0.5$

$Ln\,(1+r) = Ln\,2 = 2.3026 \times 0.30103 = 0.69315$

$$\int_0^{0.5} \frac{dx}{I_x} = \frac{0.5 \times 3}{2 \times 2^2} = 0.1875$$

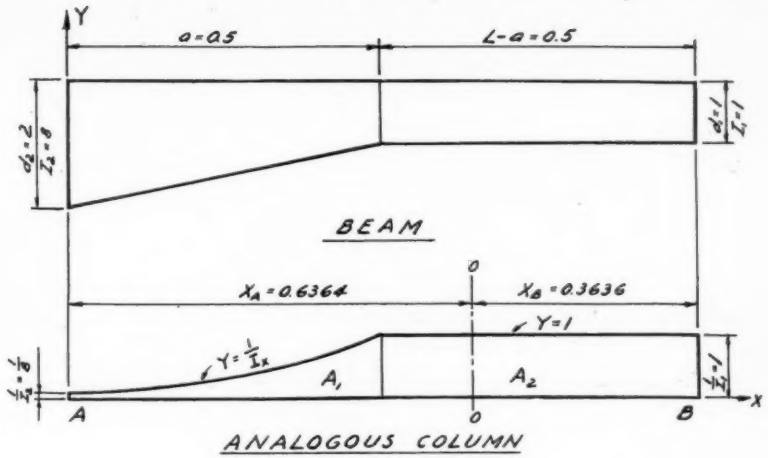


Fig. 2

$$\int_0^{0.5} \frac{x\,dx}{I_x} = \frac{0.5^2}{2 \times 2} = 0.0625$$

$$\int_0^{0.5} \frac{x^2 dx}{I_x} = -0.5^3\,(\tfrac{1}{2} - 0.69315) = 0.02414$$

$$\int_0^{0.5} \frac{x^3 dx}{I_x} = 0.5^4 \left(\frac{1 - 3 - 6}{2} + 3 \times 2 \times 0.69315 \right) = 0.00993$$

For the analogous column

Areas

$$A_2 = 0.5 \times 1 = 0.5000$$

$$A_1 = \int \frac{dx}{I_x} = \underline{0.1875}$$

$$A = 0.6875$$

Static Moments (Axis Y-Y)

$$S_1 = \int \frac{x\,dx}{I_x} = 0.0625$$

$$S_2 = 0.5 \left(0.5 + \frac{0.5}{2} \right) = \underline{0.3750}$$

$$S = 0.4375$$

Moments of Inertia (Axis Y-Y)

$$I_{1Y} = \int \frac{x^2 dx}{I_x} = 0.02414$$

$$I_{2Y} = \frac{0.5^3}{12} + 0.5 \times 0.75^2 = \underline{0.29167}$$

$$I_Y = 0.31581$$

$$X_A = \frac{S}{A} = \frac{0.4375}{0.6875} = 0.6364;$$

$$X_B = 1 - 0.6364 = 0.3636$$

$$I_o = I_Y - A x_A^2 = 0.31581 - 0.6875 \times 0.6364^2 = 0.0374$$

Carry-over factors (c) and stiffness factors (k):

$$M_A = \frac{1}{A} + \frac{x_A^2}{I_o} = \frac{1}{0.6875} + \frac{0.6364^2}{0.0374} =$$

$$1.454 + 10.827 = 12.281\ (= k_A)$$

$$M_B = \frac{1}{A} + \frac{x_B^2}{I_o} = \frac{1}{0.6875} + \frac{0.3636^2}{0.0374} =$$

$$1.454 + 3.536 = 4.990\ (= k_B)$$

$$M_{AB} = \frac{1}{A} - \frac{x_A \times x_B}{I_o} = \frac{1}{0.6875} - \frac{0.6364 \times 0.3636}{0.0374} = 1.454 - 6.187 = -4.733$$

$$c_{BA} = \frac{-M_{AB}}{M_B} = \frac{4.733}{4.990} = 0.948$$

$$c_{AB} = \frac{-M_{AB}}{M_A} = \frac{4.733}{12.281} = 0.385$$

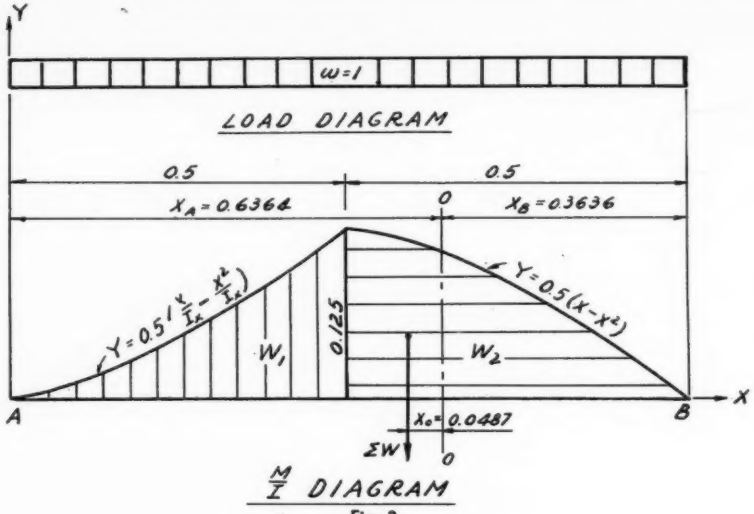


Fig. 3

Moment coefficients (f) for uniform load $w = 1$:

Areas of M/I diagram

$$W_1 = 0.5\left(\int \frac{x\,dx}{I_x} - \int \frac{x^2 dx}{I_x}\right) =$$

$$0.5\ (0.06250 - 0.02414) = 0.0192$$

$$W_2 = 2/3 \times 0.125 \times 0.5 = \underline{0.0417}$$

$$\Sigma W = 0.0609$$

Static Moment (Axis Y-Y)

$$S_1 = 0.5\left(\int \frac{x^2 dx}{I_x} - \int \frac{x^3 dx}{I_x}\right) =$$

$$0.5\ (0.02414 - 0.00993) = 0.00711$$

$$S_2 = 0.04167(0.5 + 0.375 \times 0.5) = \underline{0.02865}$$

$$\Sigma S = 0.03576$$

$$x_o = x_A - \frac{\Sigma S}{\Sigma W} = 0.6364 - 0.5877 = 0.0487$$

$$M_o = x_o \Sigma W = 0.0487 \times 0.0609 = 0.00297$$

$$M_A = \frac{\Sigma W}{A} + \frac{M_o x_A}{I_o} = \frac{0.0609}{0.6875} + \frac{0.00297 \times 0.6364}{0.0374} = 0.0885 + 0.0504$$

$$= 0.1389 = f_A$$

$$M_B = \frac{\Sigma W}{A} - \frac{M_o x_B}{I_o} = \frac{0.0609}{0.6875} - \frac{0.00297 \times 0.3636}{0.0374} = 0.0885 - 0.0288 =$$

$$0.0597 = f_B$$

Moment coefficients (f) for load P = 1 at middle of beam

Areas of M/I diagram

$$W_1 = 0.5 \times 0.25 \times 0.5 = 0.06250$$

$$W_2 = 0.5 \int \frac{x\,dx}{I_x} =$$

$$0.5 \times 0.0625 = \underline{0.03125}$$

$$\Sigma W = 0.09375$$

Static Moments (Y-Y Axis)

$$S_1 = 0.0625\left(0.5 + \frac{0.5}{3}\right) = 0.04167$$

$$S_2 = 0.5 \int \frac{x^2 dx}{I_x} =$$

$$0.5 \times 0.02414 = \underline{0.01207}$$

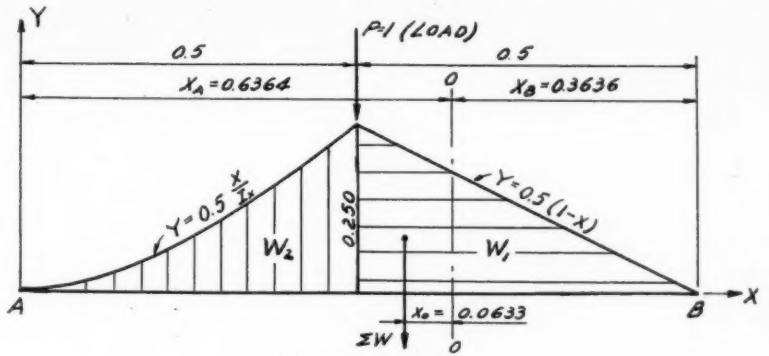
$$\Sigma S = 0.05374$$

$$x_o = x_A - \frac{\Sigma S}{\Sigma W} = 0.6364 - \frac{0.05374}{0.09375} = 0.0633$$

$$M_o = x_o \Sigma W = 0.0633 \times 0.09375 = 0.00592$$

$$M_A = \frac{\Sigma W}{A} + \frac{M_o x_A}{I_o} = \frac{0.09375}{0.6875} + \frac{0.00592 \times 0.6364}{0.0374} = 0.1364 + 0.1007 =$$

$$0.2371 = f_A$$



M/I DIAGRAM

Fig. 4

$$M_B = \frac{\Sigma W}{A} - \frac{M_o x_B}{I_o} = \frac{0.09375}{0.6875} - \frac{0.00592 \times 0.3636}{0.0374} = 0.1364 - 0.0576 = 0.0788 = f_B$$

It is seen that all results check exactly with the figures given in Table 52 of the *Handbook of Frame Constants*.

EXAMPLE 2

$r = d_2 - 1 = 0.175$

$a/r = 0.5/0.175 = 2.855$

$Ln\,(1+r) = Ln \times 1.175 = 2.3026 \times 0.07004 = 0.161$

$$\int_0^{0.5} \frac{dx}{I_x} = \frac{0.5 \times 2.175}{2 \times 1.175^2} = 0.394$$

$$\int_0^{0.5} \frac{x\,dx}{I_x} = \frac{0.5^2}{2 \times 1.175} = 0.1063$$

$$\int_0^{0.5} \frac{x^2 dx}{I_x} = -(2.855)^2 \left(\frac{0.175 \times 1.825}{2} - 0.161 \right) = 0.037$$

Data for Analogous Column

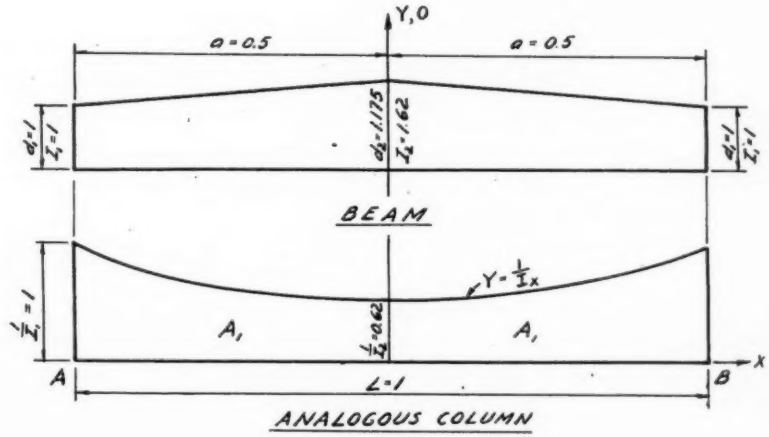


Fig. 5

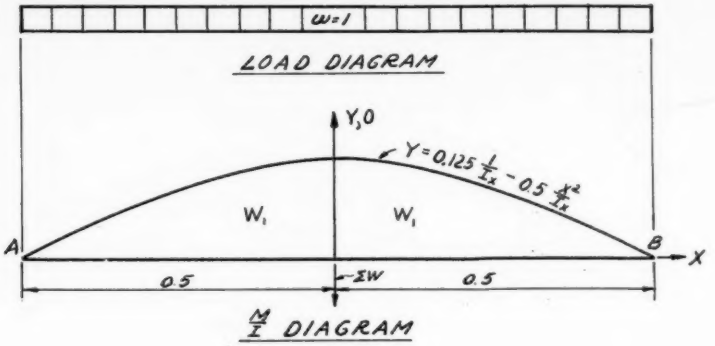


Fig. 6

$$A = 2A_1 = 2\int\frac{dx}{I_x} = 2 \times 0.394 = 0.788$$

$$I_0 = 2I_Y = 2\int\frac{x^2dx}{I_x} = 2 \times 0.037 = 0.074$$

Carry-over factors (c) and stiffness factors (k)

$$M_A = M_B = \frac{1}{A} + \frac{{x_A}^2}{I_o} = \frac{1}{0.788} + \frac{0.5^2}{0.074} = 4.65 = k_A = k_B$$

$$M_{AB} = \frac{1}{A} - \frac{x_A x_B}{I_o} = \frac{1}{0.788} - \frac{0.5^2}{0.074} = -2.11$$

$$C_{AB} = C_{BA} = \frac{-M_{AB}}{M_A} = \frac{2.11}{4.65} = 0.453$$

Moment Coefficients (f) for uniform load $w = 1$

$$\Sigma W = 2W_1 = 2\left(0.125\int\frac{dx}{I_x} - 0.5\int\frac{x^2dx}{I_x}\right) = 0.25 \times 0.394 - 0.037 = 0.0615$$

$$x_o = 0; \quad M_o = x_o \Sigma W = 0$$

$$M_A = M_B = \frac{\Sigma W}{A} = \frac{0.0615}{0.788} = 0.078 = f_A = f_B$$

Moment coefficients (f) for load P = 1 at middle of beam

$$\Sigma W = 2W_1 = 2\left(0.25\int\frac{dx}{I_x} - 0.5\int\frac{xdx}{I_x}\right) = 0.5 \times 0.394 - 0.1063 = 0.0907$$

$$x_o = 0; \quad M_o = x_o \Sigma W = 0$$

$$M_A = M_B = \frac{\Sigma W}{A} = \frac{0.0907}{0.788} = 0.115 = f_A = f_B$$

LARGY V. RUDCHENKO and WARREN E. MCNELY, Kaiser Engineers, Oakland, Calif.

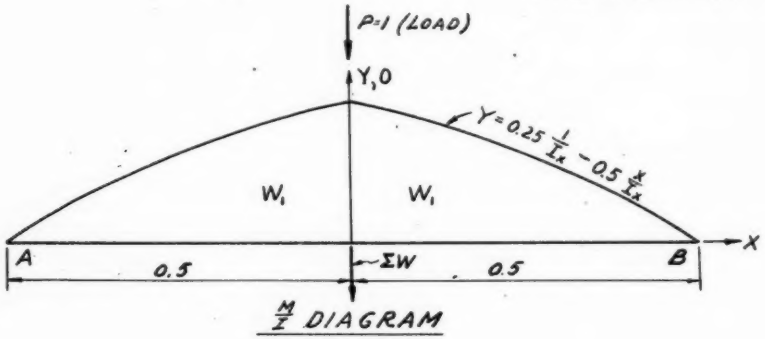


Fig. 7

Current Reviews*

of Significant Contributions in Foreign and Domestic Publications

On the autogenous healing of cement paste (in Japanese)

JIRO MURATA, *Journal of the Japan Society of Civil Engineers*, Tokyo, V. 37, No. 1, Jan. 1952, pp. 22-24

Explains the results of tests on autogenous healing of cement paste, as the preliminary test of autogenous healing of concrete. Author describes the effects of age, water-cement ratio and curing method.

On new construction methods in bridge building (in German)

G. KRALL, *Schweizer Archiv fur Angewan die Wissenschaft und Technik* (Solothurn, Switzerland), V. 17, No. 9, Sept. 1951, pp. 257-268

APPLIED MECHANICS REVIEWS Mar. 1952 (Craemer)

Describes several arched bridges of reinforced concrete bridges erected by the Melan system. Sequence of concreting is directed so as to make possible a common action of the Melan scaffolding plus first-poured parts, to support parts concreted thereafter. Creep is taken into account and some (known) formulas are quoted.

Rigid frame formulas

A. KLEINLOGEL (Translated from the German *Rahmenformeln* by I. F. MORRISON), Frederick Ungar Publishing Co., New York, 480 pp. $10

Embraces nearly all of the practical single-span types of the rigid frame. Contains formulas for reactions, shears, moments for 114 different types and shows 19 different loadings for the post and girder of each frame. Considers fixed supports, hinged supports, supports at the same and different levels, rectangular rigid frames, triangular rigid frames, rigid frames with parabolic girder and many others. 1643 figures.

Pulse techniques applied to dynamic testing

J. R. LESLIE, *Proceedings*, ASTM, V. 50, 1950, pp. 1314-1326

Describes the principles of operation of the Soniscope and its application to the testing of mass concrete for deterioration and cracking and as a device for testing laboratory specimens. The velocity of propagation of an acoustical pulse through the material being tested is measured.

Performance of concrete resurfacing in Indiana

D. W. LEWIS, *Proceedings*, Highway Research Board, V. 30, 1950, pp. 81-96

Reports results of field performance surveys of portland cement concrete resurfacing projects in Indiana built prior to 1943. Detailed data on both the design and performance are presented. It is concluded that the concrete resurfacing used in Indiana has, in general, shown potentially long service life with better resistance to pumping than full-depth slabs, but has shown durability inferior to that of full-depth pavements constructed with the same materials.

Shrinkage and plastic flow of prestressed concrete (in Japanese)

KIYOSHI OKADA, *Journal of the Japan Society of Civil Engineers* (Tokyo), V. 37, No. 2, Jan. 1952, pp. 25-28

In prestressed concrete it is well-known that the steel stress decreases gradually due not only to the shrinkage and creep of concrete, but also to the creep of steel. By use of the Davis-Glanville and Whitney laws of concrete creep theoretical solutions are pre-

*A part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 23, No. 9, May 1952, *Proceedings* V. 48. Address 18263 W. McNichols Rd., Detroit 19, Mich. Copies of articles or books reviewed are **not** available through ACI. In most cases they can be obtained direct from the original publishers. Address, when available, will be furnished by ACI on request.

sented which give the total amount of stress decrease of steel due to the conditions in prestressed concrete. Values derived were found to coincide well with later experiments.

Compressed elements in prestressed concrete (in Polish)

C. EIMER, *Inzynieria i Budownictwo* (Warsaw), V. 8, No. 6, June 1951, pp. 240-249

APPLIED MECHANICS REVIEWS Feb. 1952 (Olszak)

Author is concerned with the problem of bucking of prestressed elements when the stretched cable has separate points of contact with the element occurring at regular intervals. Moreover, such a prestressed element is compressed by an external load acting longitudinally and axially. Formulas are worked out for critical values for external compressive and for prestressing loads in various end conditions.

Prestressed concrete tanks (in Dutch)

L. H. OP TEN NOORT, *Ingenieur* ('s-Gravenhague), V. 63, Sept. 1951, pp. 65-70

APPLIED MECHANICS REVIEWS Feb. 1952

Author describes design and calculation of 375,000-gal. circular prestressed concrete sludge digestion tanks, part of the sewage treatment works at Beverwijk.

Use of high-strength wire makes possible important economies in steel and concrete, whereas the watertightness of the tanks and the permanent protection of the reinforcement against corrosion are fully provided. Details of the construction procedure are given.

Measure of the apparent density of an aggregate (Mesure de la densite apparente d'un aggregat)

PIERRE DESTABLE, *Annales de l'Institut Technique du Batiment et des Travaux Publics* (Paris), New Series, No. 168, Jan. 1951

Reviewed by PHILLIP L. MELVILLE

The measure of apparent specific gravity is of necessity inexact on account of several factors resulting from the use of a container of finite volume. Leveling off the surface, filling of the container and especially the "wall effect" will influence the results. Suggestions are made to reduce personal errors. Formula and graph are included for use in determining the apparent specific gravity of an unenclosed medium as a function of the "apparent apparent" or measured density.

Prestressed concrete in Central Africa (Le beton precontraint en Afrique Noire)

J. HUBERT, *Communication* No. B 05, International Congress on Prestressed Concrete, Ghent, Belgium, Sept. 1951, 4 pp.

Brief description of a plant at Dakar where prestressed railroad ties, beams, roof slabs and other elements are manufactured. The members, cast in movable forms, are pre-tensioned with 0.10- to 0.12-in. diameter wires up to 210,000 psi. The freshly concreted elements are cured in a heated chamber to accelerate setting.

Another plant at Dafinsi is equipped to turn out 600 ties per day. Here the wires are tensioned between anchorages.

Studies in cement-aggregate reaction XIX—The effect on mortar expansion of the particle size of the reactive component in the aggregate

H. E. VIVIAN, *Australian Journal of Applied Science* (Melbourne), V. 2, No. 4, Dec. 1951, pp. 488-494

Effect on mortar expansion of amount and particle size of the reactive component in the aggregate was examined experimentally. There is a tendency for the expansions of mortars that contain large amounts of fine reactive particles to be greater than those of mortars that contain similar or smaller amounts of larger particles. However, when all the particles are very fine (−300 BSS mesh) they do not cause the abnormal mortar expansion.

Kansas experience with concrete saw for cutting contraction joints in concrete pavements

W. J. ARNDT, *Proceedings*, Highway Research Board, V. 30, 1950, pp. 108-113

To avoid spalling and roughness at hand-formed joints, a concrete saw was used to cut them after the pavement was placed. The 2-in. deep slot thus formed was filled with thermoplastic material. Under traffic this type of joint spalled much less than hand-formed joints and had markedly improved riding qualities. Precautionary measures were necessary where membrane curing was used since erratic cracking in the continuously-placed pavement occurred at 80- to 100-ft intervals before the joints could be cut. Hand-formed grooves at 80-ft intervals were required to avoid shrinkage cracks. These were specified also on pavements cured with moist burlap and wet earth but further study may obviate their use under such curing conditions.

Bull Shoals Dam now completed

H. V. PITTMAN, *Civil Engineering*, V. 21, No. 11, Nov. 1951, pp. 26-32

Reviewed by ROBERT E. WILDE

Brief but comprehensive review of construction of Bull Shoals Dam, a Corps of Engineers project. The concrete gravity-type structure contains about 2,100,000 cu yd of concrete.

Mass concrete in the dam was divided into interior and exterior concrete, 2.5 bags of cement and 8.5 gal. of water per cu yd, and 4 bags of cement and 6 gal. of water per cu yd, respectively.

Fine and coarse aggregates production, cooling of aggregates, concrete mixing and placing are described also.

Maximum span lengths for highway deck bridges with multiple simply supported prestressed concrete girders

D. VANDEPITTE, *Communication* No. B 44, International Congress on Prestressed Concrete, Ghent, Belgium, Sept. 1951. 11 pp.

A formula is developed giving the greatest possible span length for a bridge supported by girders of a given cross section. It naturally involves, among other factors, the proportions of the cross section.

It is then determined how the cross section should be proportioned to increase the feasible span to the utmost. From this analysis a formula results giving the maximum span length as a function of the depth of the girders, the other dimensions of the cross section having been eliminated.

Application of the formula provides numerical values for the maximum span length of girders of various depths and either designed to carry American H20 loading or the live loads specified by the Belgium Highway Department.

Pile foundations

ROBERT D. CHELLIS, McGraw-Hill Book Co., New York, 482 pp., $12.50

A comprehensive treatment of the many factors affecting pile foundations. From the basic principles the author leads into the consideration of pile-driving analysis and the characteristics and capabilities of hammers and driving equipment. Then selection of pile and methods of driving, pile grouping and spacing, and structural design of piles are discussed.

The various types of precast and cast-in-place concrete piles are described with data on their manufacture and installation. Other chapters deal with wood piles, caisson-type piles and caissons, H-piles and other metal piles, sheet piling, deterioration and preservation of piles, solidification of soils, and pile load tests. Of particular interest is the chapter describing in detail 60 pile-foundation failures with causes, methods of prevention, and remedies.

Forty-eight tables and nine appendices contain a wealth of data useful to the designer, contractor and inspector. An extensive bibliography lists publications pertinent to the various phases discussed.

The compaction of concrete slabs by surface vibration: First series of experiments

R. H. H. KIRKHAM and A. C. WHIFFIN, *Magazine of Concrete Research* (London), No. 8, Dec. 1951, pp. 79-91

AUTHORS' SUMMARY

An experimental vibrator, operating on the top surface, was used to compact freshly placed concrete (mix 1:2½:5; water-cement ratio 0.60) of low to very low workability. A study was made of the effects on the compaction of slabs 18 in. thick of varying the frequency of vibration within the range up to 12 *g*, the amplitude within the range 0.004 and 0.064 in., and forward speeds of travel within the range 1 to 8 ft per minute. The density of the concrete was measured after test, while measurements were made during the experiments of the vibration occurring at various depths in the concrete.

A small variation in the workability of the concrete was found to have a considerable effect on the surcharge required and the depth of compaction achieved. Consequently the effect of the method of compaction has only been studied so far for concrete of one workability, only those tests where the compacting factor was within a narrow range around 0.80 being considered.

When the vibrator operated at constant acceleration and constant number of vibrations per foot of travel, increase of amplitude of the beam increased the depth of compaction. At constant acceleration and amplitude of the beam, slower speeds of travel gave increased depth of compaction. When both the amplitude of the beam and number of vibrations per foot run were constant, the depth of compaction was unaffected by change of acceleration.

Is scepticism towards air entrainment justified? (Scepticisme tegenover air-hulpstof?)

R. A. J. Bosschart, *Cement* (Amsterdam), No. 7-8 and 9-10, Sept. and Oct. 1951

Reviewed by J. W. T. Van Erp

Author reviews research in various countries on the effects of air-entraining agents. These data add up to a very elaborate investigation and establish some general conclusions for different makes of entraining agents. Tests in the Netherlands for instance have indicated that the flexural strength of construction joints becomes equal to that of the solid concrete, with the use of air entrainment. For lean mixes the decrease in water (and sand) increases compressive strength up to 40 percent; for heavy mixes no increase or even a small decrease in compressive strength was found. All tests show increased watertightness and resistance to freezing and chemical attack. The use of different aggregates and different test methods in various countries makes the value of comparing test data dubious, but it seems imperative to gage the quantities accurately as minute differences in the quantity of air-entraining agent may have great influence.

Testing of tilting drum mixers (Essais de betonnieres a tambour basculant)

Albert Joissel, *Annales de l'Institut Technique du Batiment et des Travaux Publics* (Paris), New Series, No. 169, Jan. 1951

Reviewed by Phillip L. Melville

A pilot study was made of the variables affecting concrete made in a tilting drum mixer. A laboratory scale and a 200-liter mixer were used. Using a gravel coarse aggregate, a river sand for fine aggregate the author reaches the following conclusions:

1. The quantity of concrete remaining stuck in the drum, after tilting, diminishes as the water content increases.

2. The inclination of the axis of rotation of the drum should be more than 30°.

3. Blades are necessary to draw the materials with them during rotation but their shape, height and angle are unimportant.

4. The speed of rotation should always be less than the critical speed $n = \frac{30}{\sqrt{r}}$ rpm (where r is the maximum radius of the drum in meters). The uniformity of the mix depends on the number of turns of the drum.

5. Mixing is easier when the mixer is less heavily loaded, but the effect of the load is small.

6. If the different components are separately loaded into the drum, they should be loaded in order from the finest to the coarsest.

7. Mixing is speeded by a more fluid mix. The most uniform concrete is obtained after about 30 revolutions. Beyond this point, segregation begins.

CRSI design handbook

R. C. Reese, Concrete Reinforcing Steel Institute, 38 S. Dearborn St., Chicago 3, Ill. 412 pp. $5.00

Presents finished designs for reinforced concrete members, giving concrete dimensions and reinforcement required. In the safe load tables, the need for charts and diagrams has been eliminated; the designer enters a table with load and span and obtains concrete outlines and reinforcing steel.

The designs given are useful for preliminary estimating, for establishing sizes and clearances, and for comparing different types of construction. While mathematically correct for the conditions stated, it does not, as no handbook can, replace the judgment of an experienced structural engineer in selecting types of structure, stresses and moment factors, in eliminating eccentricities, in providing adequate stiffness, and in obtaining satisfactory and economical structures.

Designs are based entirely on "Building Code Requirements for Reinforced Concrete (ACI 318-51)" with four exceptions; (1) the maximum positive moment in end spans was taken as $wL^2/11$ instead of $wL^2/14$ in the Code, (2) all bond computations are based on deformed bars conforming to ASTM 305-50T, plain round bars or bars not meeting ASTM 305 can not be used satisfactorily with the values given, (3) load capacities are extended beyond the 3:1 ratio of live to dead loads, and (4) square footings are based on the recommendations of F. E. Richart in "Reinforced Concrete Wall and Column Footings," ACI Journal, October and November, 1948.

The book reflects several years of intensive work by the author and will be a valuable addition to the library of every structural engineer.